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July 1995

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Engineering Reliability and Risk Analysis for Water Resources Investments; Role of Structural Degradation in Time-Dependent Reliability Analysis

by Bruce R. Ellingwood, Johns Hopkins University

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Prepared for Headquarters, U.S. Army Corps of Engineers

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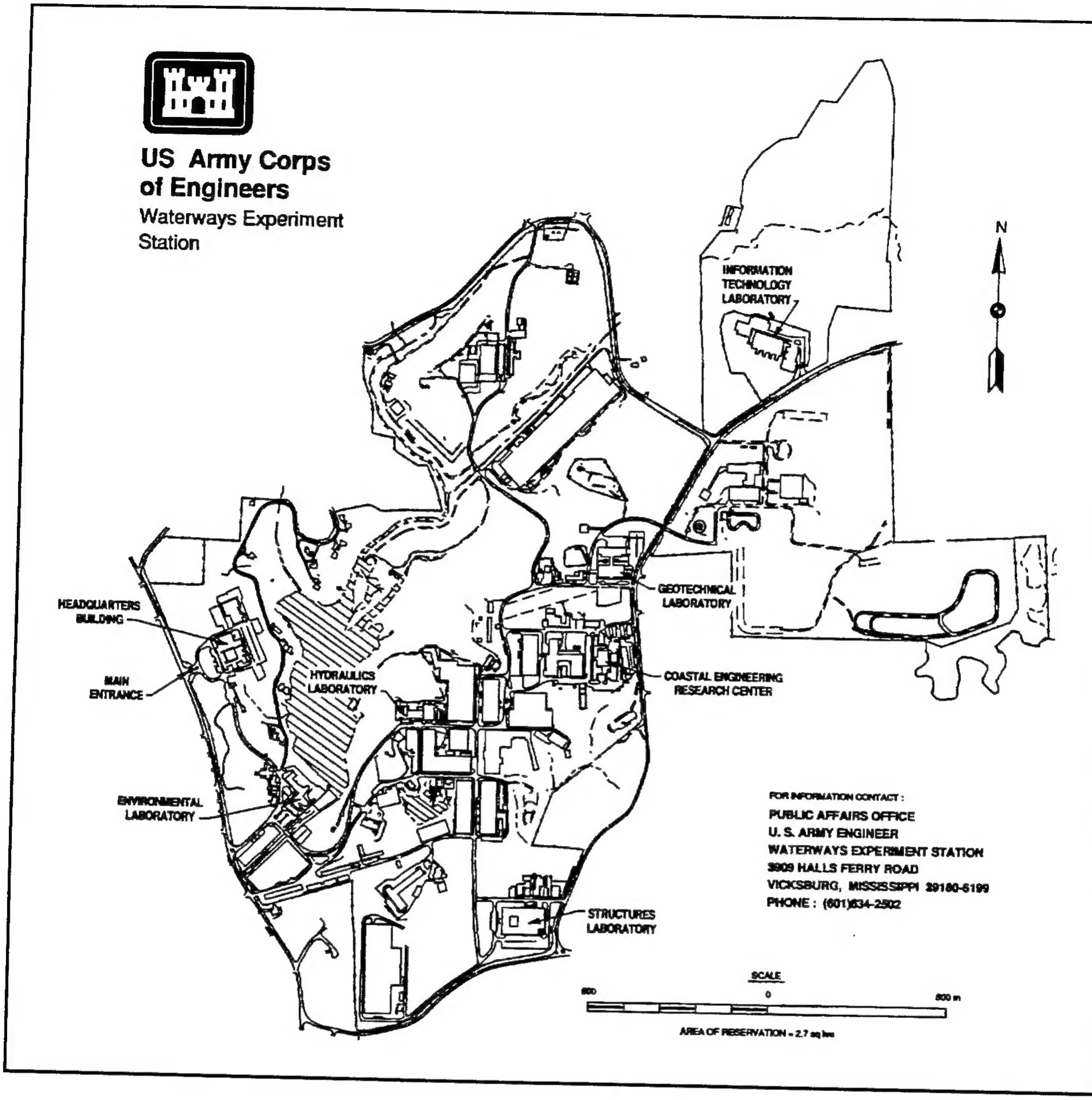

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PREFACE

This report describes a summary of the role of structural degradation in time-dependent reliability analysis and presents recommendations for developing methods applicable to U.S. Army Corps of Engineers (USACE) civil works structures. The work was funded under the Civil Works Research and Development Program, Risk Analysis for Water Resource Investments, at the U.S. Army Engineer Waterways Experiment Station (WES). Dr. Bruce R. Ellingwood at Johns Hopkins University performed the work under USACE contract number DACW39-93-M-6302 monitored by Dr. Mary Ann Leggett of the Scientific and Engineering Applications Center, Computer-Aided Engineering Division (CAED), Information Technology Laboratory (ITL), WES. The work was coordinated by Messrs. Jerry Foster and Don Dressler of the Engineering Division, Directorate of Civil Works, Headquarters, USACE. The work was performed under the general supervision of Mr. H. Wayne Jones, Chief, CAED, ITL, and Dr. N. Radhakrishnan, Director, ITL.

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1. INTRODUCTION

The investment in the nation's infrastructure is at risk from deterioration compounded by inadequate inspection and maintenance. In the last several years, this problem has been recognized (Bridge, 1987; ASCE, 1991; Suprenant, et al, 1992)), and is being addressed by some federal, state and local agencies. As one example, the Intermodal Surface Transportation Efficiency Act (ISTEA) of 1991 requires issuance of regulations for State development, establishment and implementation of systems for managing (1) highway pavements, (2) bridges, (3) highway safety, (4) traffic congestion, and (5) public transportation facilities, and (6) intermodal transportation facilities and systems. (Federal Register, Vol 57, No. 107, June 3, 1992). The technology required to maintain these management systems is being developed.

The nation's inland navigation system plays a significant role in the overall health of the nation's economy, providing a mode of transport for natural, agricultural and industrial products. Structures that are part of the inland navigation system are deteriorating. Many of these structures were constructed prior to the 1940's. Such structures often cannot be replaced without significant economic dislocations, and a large investment in maintenance is required to keep them operational. Management and life extension of navigation facilities and structures are becoming important in the current economic climate with limited public funds and a declining tax revenue base. One of the most critical needs in the effort to improve policies for management of aging navigation infrastructure is to develop technologies to facilitate the assessment and rational prioritization of maintenance actions, with due regard to the fact that budgets available for such activities usually are limited. At present, such technologies do not exist in useable form. Methodologies for selecting and ranking investments concerning infrastructure rehabilitation need be economically motivated, but require significant technical engineering input, particularly since the decisions impact public health and safety as well as economic productivity. Such decision tools must reflect recent advances in the prediction of structural performance in service, measurement technology, and minimum life-cycle cost analysis (DeKraker, et al, 1982; Siemes, et al, 1985; CIB 1987; Rostam, 1993). Comparisons among alternate strategies should be made using probabilistic and statistical methods, where uncertainties can be dealt with explicitly and systematically (ASME, 1992; Cesare, et al, 1993).

A large portion of the navigation infrastructure in the United States is constructed from steel or reinforced concrete. Concrete and steel are durable materials when properly placed and maintained. However, their structural properties and performance in service can be negatively impacted over time by faulty design, unsuitable materials, improper workmanship, exposure to aggressive environments, excessive structural loads, and accidental loading conditions and abuse. Structures are subject to a phenomenon known as aging, in which the material and structural properties change over time (Naus, 1986; Hookham, 1991; Shah and McDonald, 1993). Some of these aging effects are relatively benign. Others may cause the load-carrying capacity of a structure to decrease and may increase the risk to public safety if not properly controlled. Most of the more significant environmental stressors and mechanisms of attack that may lead to aging and deterioration of steel or concrete structures have been identified qualitatively (Al-Tayyab and Khan, 1988; ACI 222, 1985; Berger, 1983; Clifton and Knab, 1989; Komp, 1987; Lauer, 1990; Somerville, 1986; Tuutti, 1982). However, only in a few instances have their quantitative effects been incorporated in life-cycle performance modeling of civil infrastructure.

Numerous sources of uncertainty complicate the evaluation of aging effects on the residual strength of steel or concrete structures. Uncertainties arise from (1) inherent randomness in structural loads, engineering material properties and strength degradation mechanisms; (2) differences in design codes and standards for structural components of different ages; (3) lack of in-service measurements and records; (4) limitations in available models for quantifying time-dependent material changes, damage accumulation, and their contribution to structural strength; (5) inadequacies in nondestructive evaluation (NDE) technologies; and (6) shortcomings in existing methods for rehabilitation and repair. Modern statistical analysis methods are required to develop the needed probabilistic models from the often limited data available.

A management system for navigation infrastructure should provide a set of criteria, rules and quantitative tools that can be used to evaluate structures and components for fitness for continued service. The evaluation of aging concrete or steel structures for continued service should provide quantitative evidence that their strength is sufficient to withstand future extreme events within the proposed service period with a level of reliability sufficient for public safety without wasting public resources. Criteria on fitness-for-service should be useable under difficult field conditions; minimize the effects of operator influence; be effective with badly deteriorated surfaces; and be economical.

2. OBJECTIVES AND SCOPE

This report presents the results of a preliminary investigation of the feasibility of incorporating structural degradation into time-dependent reliability analysis of navigation infrastructure on inland and coastal waterways. An appraisal is presented of methodologies for engineering decision that might enable the owner/operator of a navigation facility to perform condition assessments, determine its economical remaining service life, and schedule routine maintenance and repair so as to minimize total operating costs without endangering public safety. The report is developed around the following specific research objectives:

1. Identify mathematical models to evaluate degradation in strength of navigation structures over time in terms of initial construction conditions, service load history, and aggressive environmental factors.

2. Develop a methodology to assess the probability that structural capacity has degraded below a specified level, taking into account initial conditions of the structure, service load history and structural aging, and deterioration.

3. Examine the role played by peridic inspection, nondestructive evaluation and maintenance in maintaining reliability and minimizing overall costs during a projected service period.

This feasibility study is based on a review, synthesis and interpretation of existing procedures and data in the structural engineering, material science, and structural reliability literature. It provides a prospectus for subsequent research on reliabiity-based evaluation of navigation structures, some of which is initiating (Padula, et al, 1994; "Reliability", 1994).

3. MATHEMATICAL MODELING OF STRUCTURAL DEGRADATION

The use of mathematical modeling to determine structural performance in service requires a knowledge of the initial conditions of the structure (material strengths, dimensions) and an understanding of how aggressive environmental stressors impact its performance over its service life.

3.1 Degradation Mechanisms

Predictive models that evaluate the change in strength of a steel or concrete structure over time in terms of initial conditions, applied load history, and some parameterization of the aggressive environment should be based on structural mechanics principles and need to be calibrated to actual data. For some mechanisms of strength degradation (e.g., stable crack growth in weldments subjected to cyclic loads), the mechanics of deterioration are reasonably well established and predictable through fracture mechanics principles (e.g., Barsom and Rolfe, 1987; CIB, 1987). In other cases (e.g., freeze-thaw cycling, sulfate attack, expansive aggregate reactions), the behavioral models are less certain and may have been developed from laboratory tests that are atypical of in-service conditions (Clifton, 1991).

Corrosion in steel, fatigue/crack growth in weldments, anchorage failures of metal devices embedded in concrete, and freeze-thaw cycling and abrasion in concrete are known to have a significant impact on time-dependent performance of navigation structures on inland waterways and in coastal environments. Structurally significant components that are particularly affected by such effects include vertical and horizontal lockwall surfaces, lock agates and anchorages for equipment and moveable components, and beams and columns. Restricting attention to these components will be sufficient to demonstrate the feasibility of time-dependent reliability analysis.

Mechanisms such as sulfate attack and alkali-aggregate (alkali-silica) reactions in concrete, both of which give rise to expansive products and internal tensile stresses that may result in cracking, are not as likely to cause significant aging damage in navigation infrastructure. Such mechanisms are most commonly observed in the Western United States. Several of these mechanisms are not easily detectable, and have the potential to cause widespread concrete deterioration and structural strength reduction. Leaching, or deterioration of the surficial concrete by dissolution of soluble components of hardened cement paste in contact with water, has not been reported to be a problem in navigation structures. Creep, shrinkage and settlement, and thermal exposure damage are believed to pose a minimum risk. However, these degradation mechanisms can be considered at a later time for specific components and structures, if necessary.

Most degradation processes, including corrosion of steel or disintegration of concrete, require water either to initiate the chemical reaction or to saturate the concrete before major deterioration occurs.

Weathering and Freeze-Thaw Damage in Concrete

Weathering causes concrete to deteriorate from cycles of freezing and thawing and from restrained expansion and contraction due to variations in temperature. Most damage to hardened concrete is likely to occur in zones of repeated wetting and drying. The permeability of the concrete is perhaps its most important attribute for mitigating such damage (Philleo, 1987).

Any concrete in a freezing environment that may become saturated with water, or nearly so, is susceptible to freezing and thawing damage. Such damage is one of the most common causes of deterioration of mature concrete. Freeze-thaw damage occurs when the temporature is below a critical temperature at the same time that satuation exceeds a critical level. Flat horizontal surfaces where water may accumulate and vertical surfaces exposed to repeated cycles of wetting and drying where temperature conditions are most extreme are particularly susceptible. Repeated cycles of freezing and thawing has the most pronounced effect on the tensile strength of concrete, which deteriorates with cracking. The resistance to damage by freeze/thaw is mainly a function of the capillary pore structure of the cement, the moisture content of the concrete and the rate of freezing, assuming that the aggregates are not involved. The phase change from water to ice is accompanied by a volume increase of approximately 9 percent. If the volume of water in the capillary pores of the concrete exceeds 91 percent of the pore volume, expansion during freezing cannot be accommodated, hydraulic pressures resulting from this volumetric increase cannot be prevented, and cracking of the concrete will occur. If the capillary pores are fully saturated, 9 percent expansion may exert an internal pressure as high as 100 MPa. However, the concrete does not have to be fully saturated; critical saturation depends on the concrete mix, and is affected by the distribution of aggregrate sizes, water/cement ratio, and generally by the permeability. A level of 85% saturation is sufficient for freeze-thaw damage to occur. Moreover, damage is not limited to the cement paste; aggregates can undergo damage as well (Clifton, 1991). The critical temperature is somewhat less than 0°C (32°F) because of impurities in the water; a recommended value is -5°C (23°F). (Neville, 1981).

Air entraining admixtures improve the durability of concrete and reduce such damage by providing uniformly distributed pressure-relief voids throughout its volume, provided that the air bubbles do not eventually become filled with water if no drying can take place. These admixtures are chemically inert after proper concrete curing. For severe exposures, a recommended air content for frost-resistant concrete would be roughly 4 - 5% for maximum aggregates sizes of 25-76 mm (1 - 3 inches). Most of the concrete used in existing inland waterway navigation facilities has not been air-entrained.

Erosion due to abrasion/cavitation/scour

Surficial erosion due to abrasion is caused by repeated rubbing and grinding of particles on the concrete surface, and might be found on slab or wall elements exposed to moving water, water-borne rocks or other debris, or barge traffic. Soluble constituents of the cement matrix are washed out and the gradual wearing away of the surface exposes the aggregates. Abrasion typically leaves a relatively smooth surface. Cavitation is caused by impact forces that result from the collapse of vapor bubbles in rapidly flowing water. Damage due to cavitation is manifested by a rough and pitted surface, in which large quantities of the concrete may be removed. Cavitation damage, once started, accelerates because damaged rough surfaces are more susceptible to cavitation forces. Water velocities in excess of 12 m/s can produce cavitation damage. The damage normally begins on the downstream edge of concrete joints. Both abrasion and cavitation damage are accelerated by freeze-thaw cycling, which causes the outer layers of the concrete to disintegrate. In a lock structure, the effect of this synergism is likely to be most pronounced at the mean elevation of the upper pool.

Corrosion

Corrosion is an electrochemical reaction, requiring an electrolyte in contact with the metal. Metallic surfaces exposed to oxygen (atmospheric or dissolved in water) form a protective oxide passive layer that protects against initiation of corrosion; this layer can be damaged by mechanical actions (stresses, abrasion), allowing corrosion to initiate. By one estimate (Schweitzer, 1987), the direct and indirect cost of corrosion in the United States is $80 billion annually. There are many different forms of corrosion (Berger, 1983). Uniform corrosion is among the most common forms, and is characterized by a progressive and more or less uniform thinning of an entire section with time. Pitting corrosion is highly localized, is initiated by local acidity, inhomogeneities or small defects in the material, and is self-propagating and often difficult to detect. It is particularly significant in pressurized containers, where it leads to through-thickness cracking and leakage. Stress-corrosion cracking is corrosion accelerated by external or residual stresses and by cyclic stresses that disrupt the protective passive layer, giving rise to corrosion fatigue. In hydraulic structures, general corrosion and corrosion-fatigue are believed to have the most significant impact on structural behavior.

Portland cement concrete protects embedded steel components, such as reinforcement and anchorages, against corrosion by providing a protective oxide layer on the surface of the steel because its alkalinity (pH between 12 and 14) (Swiat, 1993). Depassivation and breakdown of this protective layer can occur by carbonation (ingress of carbon dioxide) or by penetration of chlorides. Such processes occur slowly in concrete with low permeability. Carbonation is a reaction between the calcium hydroxide in the cement and carbon dioxide which reduces the cement pH and depassivates the surface of embedded steel. It normally is a slow process, and is lower if the concrete is fully saturated with water, which impedes the diffusion of carbon dioxide. Penetration of chloride ions to the steel surface destroys the passivity even at high pH values and makes the surface susceptible to general or pitting corrosion. Water-soluble chloride ions are responsible for corrosion. Structures in contact with seawater or chloride-containing ground water are particularly susceptible; however, brackish water found in many inland waterways also may accelerate corrosion. Chlorides also can be present in the aggregates or mix water, especially if coastal aggregates or seawater are used in the concrete, and sometimes have been added to concrete as constituents of cure-accelerating admixtures. Cracks in the concrete due to the above environmental stressors described above may accelerate the initiation of corrosion. Once corrosion of reinforcement initiates, its rate is determined primarily by the availability of moisture and oxygen. If the concrete is water-saturated, the solubility of oxygen is reduced and the corrosion rate of steel is small. The corrosion products of steel occupy a volume of from 3 to 7 times the volume of the original steel. The stresses that result from this expansion lead to cracking, spalling and delamination of the concrete cover. Once corrosion has initiated, these processes occur rapidly.

If corrosion is the determining factor in the service life of a structure, the useful service life is the sum of an induction period, during which carbonation or chloride ion penetration of the concrete cover to the reinforcement occurs, and an active corrosion or deterioration period, during which loss of reinforcement area and expansive reaction occurs. Since the induction period usually is substantially longer than the active period, many researchers have used it as the basis for service life assessments (e.g., Tuutti, 1982; Vesikari, 1988; Clifton, 1989; 1991). The rate of active corrosion depends on the environment. The mechanisms of corrosion described above suggests that its effects are likely to be most severe in regions of alternate wetting and drying. This observation has been

confirmed in field studies (Borstov, 1983).

Failure of Embedded Metal Anchorages

The strength of anchorages depends on the initial strength of the concrete and angle of pull. Failures arise from deterioration of concrete in the vicinity of the anchorage, often coupled with corrosion, which may lead to pullout of load-carrying anchorages.

Fatigue

Fatigue is a process used to describe the accumulation of damage due to fluctuations in load over time at stresses or strains that generally are substantially less than would cause failure under monotonic loading. Although the exact nature of fatigue is not fully understood at the phenomenological level, it is caused by localized reversals of inelastic action at local inhomogeneities or at notches, holes, weld undercuts, and similar stress raisers. The fatigue process involves an initiating phase and a subcritical crack growth phase, followed by crack instability of fast-fracture. The point of demarcation between crack initiation and growth is somewhat arbitrary but usually is defined as the point at which the crack or flaw becomes visible or detectable by nondestructive evaluation (NDE). Different methods are used to analyze the initiation and growth phases, as described subsequently (Committee, 1981; Yao, et al, 1986).

The decrease in strength with increasing number of cycles, often is expressed by an S-N curve, relating elapsed cycles, N, to stress or strain range or amplitude, S. In concrete, cycling at 55% of the specified initial strength of concrete leads to failure in approximately 10^7 cycles. Fatigue loading causes microcracks to develop in the cement paste, which spread throughout the concrete during subsequent repeated loading. Fatigue problems generally are slow to develop in concrete and typically become obvious before significant damage or failure can occur. However, the widening cracks may expose the interior of the concrete to other aggressive substances. Cyclic loads can occur at equipment anchorages and supports and other regions where operating equipment is located, which can cause some degradation in strength. Fatigue of steel reinforcement is not considered a problem because the stresses in the reinforcement at service load levels are usually below the fatigue threshold for Grade 60 reinforcement (approx 140 MPa). In steel structures and components, fatigue/fracture are significant problems in welded components, where residual stresses, stress concentrations and careless repair operations may aggravate the problem. Fatigue damage can be accelerated in the presence of active corrosion, a synergistic effect known as corrosion-fatigue. Corrosion-fatigue is affected mainly by stress or strain range, stress raisers, residual stress, cycling frequency and cathodic potential; the fatigue life may be reduced by as much as 50-75% (Jaske, et al, 1978).

3.2 Mathematical models of degradation

3.2.1 Transport Processes

The environmental stressors described above give rise to degradation in the concrete and steel with time. Mathematical models of structural degradation of concrete over time can be formulated in terms of transport processes that describe the intrusion of degrading agents into the concrete (Pommersheim and Clifton, 1990). Such processes can be convection-controlled (governed by differences in pressure, or hydraulic gradient) or diffusion-controlled (governed by differences in

concentration). Convection can be an important mechanism for intrusion of a penetrant unless the pores are small or there is no pressure gradient. If the concrete is cracked or spalled, convection is likely to control the intrusion of deleterious substances. In addition, sorption or reaction processes occur within the
concrete.

The mathematical model of the process by which aggressive substances penetrate the concrete depends on the governing mechanism. A one-dimensional formulation often is sufficient to gain insight, since the thickness of the component usually is much smaller than its plan dimension. Consider, for example, a segment of wall or slab with thickness, W, illustrated in Figure 3.1*. If the penetration is diffusion-controlled, the concentration, c(x,t), is described by:

$$\frac{\partial c}{\partial t} = D \frac{\partial^2 c}{\partial x^2} \tag{3.1}$$

in which D = diffusivity coefficient. If the process is convection-controlled,

$$\frac{\partial c}{\partial t} = +h \frac{\partial c}{\partial x} \tag{3.2}$$

in which h = convection or transfer coefficient. Finally, if a combination of processes governs,

$$\frac{\partial c}{\partial t} = D \frac{\partial^2 c}{\partial x^2} + h \frac{\partial c}{\partial x} \tag{3.3}$$

Numerical solutions of these equations for various deterioration mechanisms can be used to relate the concentration to depth of potential degradation as a function of time.

The depth of deteriorated concrete or steel often can be modeled to an acceptable approximation by,

$$X(t) = C\,(t - t_i)^{\alpha} \tag{3.4}$$

in which t = time, t_i = induction or initiation time required to activate the process, C = rate parameter, and α = time-order parameter. The parameters C and α must be determined from experimental data. (In a time-dependent structural reliability analysis, C and t_i are modeled as random variables.) For a diffusion-controlled process, $\alpha = 1/2$ (carbonation, diffusion of chloride corrosion of steel reinforcement). For some attack mechanisms, $\alpha > 1.0$ (e.g., sulfate attack of concrete). One must be cautious in using the results of accelerated aging tests to determine C and α, however, as the aging mechanisms may not scale properly from the laboratory to the prototype

* Tables and Figures appear at the end of each chapter.

(Clifton and Knab, 1989).

Table 3.1 summarizes average corrosion parameters for carbon and weathering steels in several environments (Komp, 1987). These values were determined from tests of small specimens, and some error may result from extrapolating these values to structural members. Constants C and α are such that X(t) is measured in μ-m when t is measured in years. These parameters have been used in reliability-based evaluation of bridge deterioration (Kayser and Nowak, 1989) and to devise bridge inspection strategies (Sommer, et al, 1993).

Structural capacity can be related to X(t) for a given behavioral limit state of interest. Consider the simple case illustrated in Figure 3.1, in which a degradation "front" described by X(t) penetrates a structural component, such as a shear wall, with thickness W. We assume that there is no residual load-carrying capacity in the degraded (shaded) portion of the wall; thus, when X(t) reaches W, the wall loses all capacity to carry load.

The impact of such degradation depends on the nature of the limit state. If, for example, the limit state in the wall is one of tension or compression, the undamaged area and strength are proportional to 1 - X(t), i.e.,

$$R(t) = F_{cr} A(t) \tag{3.5a}$$

$$A(t) = A_o(1 - X(t)) \tag{3.5b}$$

in which F_{cr} = limiting material strength and A_o = initial area per unit length of wall. On the other hand, if the limit state is one of flexure, we have

$$R(t) = F_{cr} \;\; (I(t)/y) \tag{3.6a}$$

$$I(t) = I_o(1 - X(t))^3 \tag{3.6b}$$

in which I_o = initial moment of inertia per unit length and y = distance to the extreme fiber in flexure. In the case of flexure, small errors in estimating X(t) will have a more significant impact on predicted flexural strength, R(t), in a relative sense than errors in estimating F_{cr} from in situ testing. As a result of degradation, e.g. due to corrosion, the governing limit state can change during the service life of the structure (Roth, et al, 1992), creating an unanticipated and potentially dangerous situation. As will be described in Section 4.2, the probability distribution of X(t) will play a key role in the condition assessment analysis.

Freeze-thaw damage cannot be modeled by the models in Eqns 3.1 - 3.4. Existing models for assessing freeze-thaw damage in the literature generally are unsatisfactory for making quantitative service life predictions (Clifton and Knab, 1989). Current test procedures are aimed more at suitability in a qualitative sense of concrete mix constituents. Cycles of freezing and thawing affect mainly the tensile strength of the concrete. Recent work has indicated that this dependence can

be expressed as ("Reliability", 1994),

$$f_t(n) = f_{t_o}/n^b \tag{3.7}$$

in which f_{t_o} is the initial tensile strength of the concrete, measured by the modulus of rupture, and b is an experimental constant. To utilize this relation, the number of freeze-thaw cycles, n, must be estimated; this estimation is nontrivial.

The choice of temperature range and saturation of concrete are difficult aspects of assessing freeze-thaw damage. In temperate climates, the winter months usually provide the governing condition, because the structural actions are proportional to the difference between interior and exterior temperatures, which are greatest in winter. Most thermal analyses ignore the transient heat flow aspects and assume that steady-state conditions prevail. This is a significant shortcoming in the analysis of concrete, which is a poor thermal conductor. Thermal gradients in a thick concrete wall are not as severe as would be predicted from a steady-state analysis. The ingress of moisture by seepage and capillary action also is difficult to predict analytically.

3.2.2 Fatigue and Fracture

Fatigue relations are empirical to a degree, with parameters determined generally by testing small specimens under cyclic load. The primary load parameter affecting fatigue is the stress (or strain) range, $\Delta S = S_{max} - S_{min}$. Other factors including mean stress (or stress ratio S_{min}/S_{max}), cyclic frequency and environment also may affect behavior. Fatigue life is defined in a number ways: as time (or number of cycles) to complete fracture of the specimen; as time required for a specified increase in specimen compliance; or as time to repairable cracking.

Structural failures due to fatigue/fracture are modeled in two distinct stages (Committee, 1981; Yao, et al, 1986). The first stage is that required to initiate a crack in a defect-free component. The number of cycles, N_i, (or time) required to initiate a (visible) detectable crack under constant amplitude loading can be described by the well-known S-N relation between stress and cycles (or strain and cycles when the loading is deformation rather than load-controlled):

$$N_i \, \Delta S^m = C \tag{3.8}$$

in which ΔS = applied stress (strain) range and m and C = experimental constants. Eqn 3.8 is sometimes referred to as the Basquin equation and is considered valid in the high-cycle region (greater than approximately 10^5 cycles), where the stresses remain entirely in the elastic range. A more general model is the Coffin-Manson equation (discussed in Committee, 1981):

$$\Delta\varepsilon/2 = (\sigma_f'/E) \; (2N)^b + \varepsilon_f' \; (2N)^c \tag{3.9}$$

in which $\Delta\varepsilon$ = strain range, E = modulus of elasticity, and the other terms are empirical constants. The first term is equivalent to Eqn 3.8, expressed in terms of elastic strain; the second term

dominates in the low-cycle range, where repeated inelastic cycling occurs. S-N curves depend on the material and fatigue-critical structural detail. Several typical S-N curves are illustrated in Figure 3.2. The curves labeled AASHTO B,C,D, and E are based on fatigue tests of welded details found in highway bridges (Keating and Fisher, 1986); curves API-X, AWS-X-Modified and DEn are based on the American Petroleum Institute, American Welding Society, and United Kingdom Department of Energy design guides, respectively (reported in Murthy, et al, 1994). These S-N curves are based on air-fatigue tests; note that exponent $m \approx 3$ in all cases. In a corrosive environment, m tends to increase to the range $m \approx 3.5 - 4.0$.

When the load amplitudes vary in time, the time (or cycles) to failure is determined from a cumulative damage law. Such laws relate fatigue behavior under variable amplitude loading to the known behavior under constant amplitude loading described in Eqs. 3.8 or 3.9. The most common of these laws is the Palmgren-Miner hypothesis (Miner, 1945), which postulates that damage accumulates linearly simply as a function of the number of cycles at a particular stress level. Under variable amplitude loading, damage accumulation D(t) is described by,

$$D(t) = \sum_{i=1}^{M(t)} \Delta D_i = \sum_{i=1}^{M(t)} [N(\Delta S_i)]^{-1} \tag{3.10}$$

in which $M(t)$ = number of load cycles in $(0,t)$, ΔD_i = increment of damage in cycle i, and $N(\Delta S_i)$ = number of cycles to failure under constant amplitude loading ΔS_i, determined from Eqs 3.8 (or 3.9). If the load history consists of k discrete load amplitudes, Eqn 3.10 takes the more familiar form,

$$D(t) = \sum_{i=1}^{k} n_i/N(\Delta S_i) \tag{3.11}$$

in which n_i = number of cycles in the load history at stress level ΔS_i, and $\Sigma n_i = M(t)$. Failure is said to occur when $D > 1.0$. This simple damage hypothesis does not account for stress sequence effects on fatigue life. However, more complex rules do not provide consistently better results (Committee, 1981).

Once a crack of size a has initiated, its subsequent growth rate can be predicted by the Paris equation (Barsom and Rolfe, 1987),

$$\frac{da}{dN_p} = C\,(\Delta K)^m \tag{3.12}$$

if the structure is elastic, in which C and m are experimental constants; note that C and m in Eqs 3.8 and 3.10 are not the same. The stress intensity factor, ΔK, is defined as,

$$\Delta K = Y(a)\,\Delta S\sqrt{\pi a} \tag{3.13}$$

in which ΔS = far-field stress range, a = crack size, and Y (a) = finite geometry correction factor, dependent on the shape of the flaw and the relative flaw size, a/W. If the stress history is known, Eqn. 3.11 can be integrated to determine flaw size as a function of elapsed cycles or of time. Refinements to these equations that take into account the mean stress (strain) do not measurably improve their predictive performance. Procedures also are available for modeling nonlinear behavior (Barsom and Rolfe, 1987).

The total fatigue life, N_t, of the component is,

$$N_t = N_i + N_p \tag{3.14}$$

The relative contributions of N_i and N_p to N_t depend on the load spectrum, material characteristics, initial condition of the structure. If the structure is unflawed and the stress range is low, $N_i > N_p$; indeed, in many studies, N_p is neglected and $N_t = N_i$. On the other hand, many welded structures contain initial flaws (lack of fusion, penetration), and in such components $N_i = 0$. The constants in Eqs. 3.8 and 3.12 are determined experimentally, and both relationships exhibit significant statistical scatter (Provon, 1987; Ortiz and Kiremidjian, 1988). When cycling occurs in an aggressive environment (corrosion-fatigue), the constants C and m in these equations are different from what would be observed by cycling in air, the exponent m decreasing in Eqn. 3.8 while increasing in Eqn 3.12. The relation between damage accumulation and stress is highly nonlinear in both cases, which makes it difficult to deal with corrosion-fatigue by simply applying a penalty factor to air-test data. Additional fatigue test date for the environmental conditions expected would be desirable.

Table 3.1 - Corrosion Parameters in Eqn 3.4

Environment	Carbon steel		Weathering steel	
	C	α	C	α
Rural	34	0.65	33	0.50
Urban	80	0.59	51	0.57
Marine	71	0.79	40	0.56

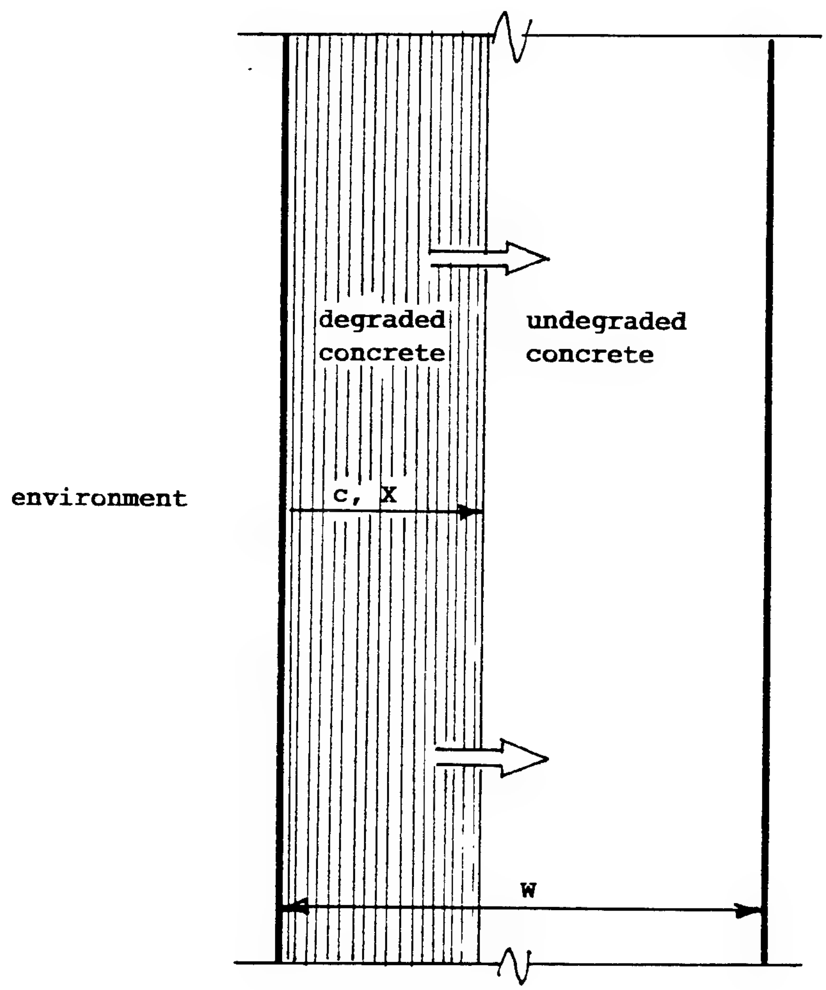


Figure 3.1 - Illustration of degradation penetrant

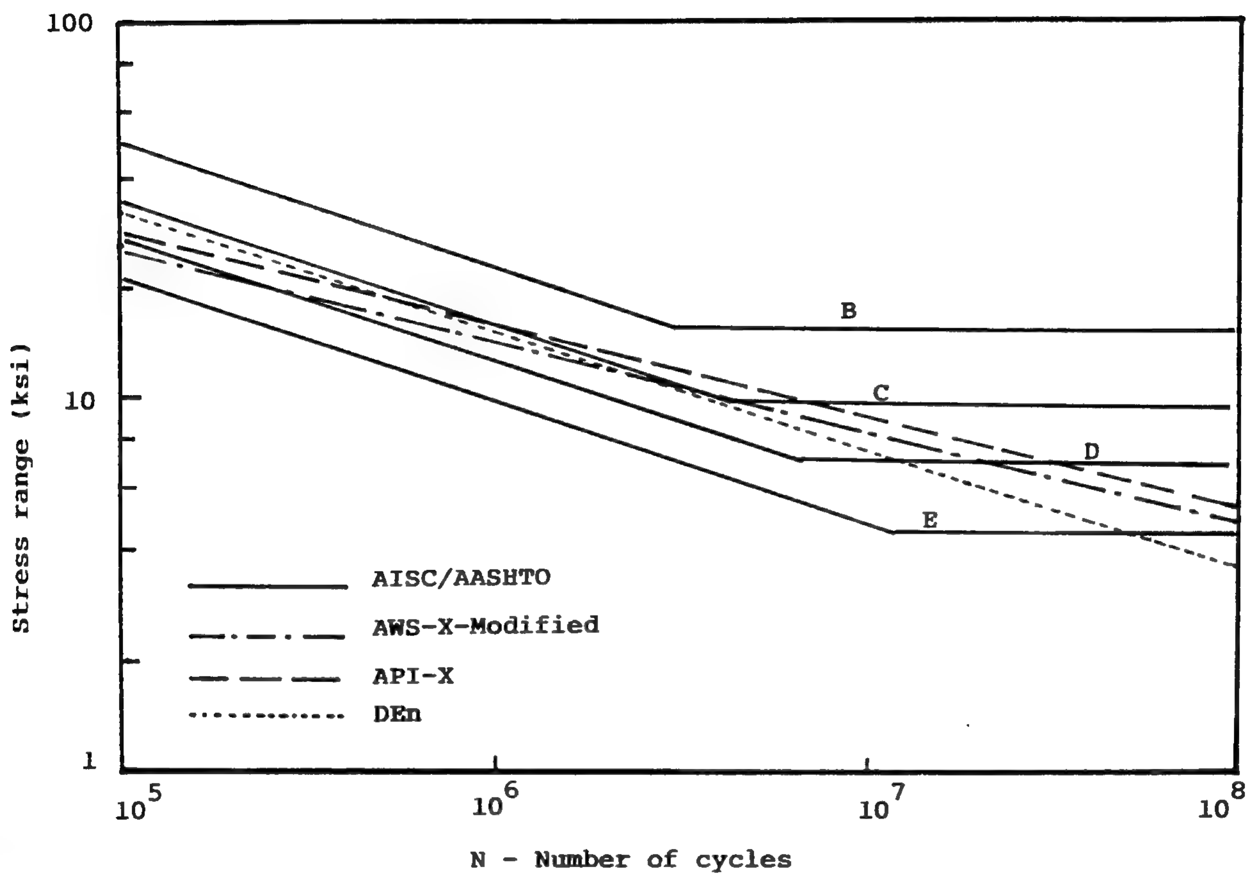


Figure 3.2 - Typical S-N Curves for fatigue design

4. TIME-DEPENDENT RELIABILITY ASSESSMENT

Time-dependent reliability analysis methods provide the framework for dealing with the uncertainties involved in performing condition assessments of existing structures and in determining whether in-service inspection/maintenance is required to maintain relibility and performance at the desired regulatory level. Statistical data to describe the time-dependent characteristics of the loads acting on the facility and its structural capacity are required to utilize the power of time-dependent reliability analysis as a decision tool.

Structural load models and descriptive statistics have been gathered in previous research to develop probability-based limit states design procedures (e.g., MacGregor, et al, 1983; Ellingwood and Mori, 1993). Some of these models are applicable to navigation facilities. However, to perform reliability assessments of new or existing structures that may age in time, the structural resistance over time also must be described statistically. In the absence of full-scale monitoring of structural performance, this resistance must be obtained from the mathematical models of those degradation mechanism(s) present, as summarized in Section 3. None of the reliability analyses performed as part of the basis for probability-based limit states design procedures (e.g., Ellingwood and Galambos, 1982) have taken time-dependent behavior of structural resistance into account. Moreover, only limited data on time-dependent properties of resistance are available (Mori and Ellingwood, 1993).

Section 4 provides an overview of probabilistic methods that have proved useful in structural reliability assessments of new and existing structures. A critical appraisal is made of those methods in terms of their potential application to navigation structures.

4.1 Statistical Models of Operating and Environmental Effects

Structural loads are random in their temporal occurrence, spatial variation and intensity. Although loads are modeled most accurately as random (stochastic) processes, such models are difficult to handle mathematically in structural reliability analysis and to implement with the data that are available from operating or environmental records. One of the challenges in structural load modeling is to develop simple random variable characterizations of stochastic load processes that capture the essential features of their temporal and spatial variation without the attendant mathematical complexities.

Discrete Load Models. Rare events such as accidental impact and extreme environmental events such as earthquakes can be modeled as a sequence of short-duration loads (impulses) occurring randomly in time. One of the simplest pulse process models is illustrated by the sample function in Figure 4.1a. The occurrence in time of the loads (impulses) is described by a Poisson process, with mean (stationary) rate of occurrence, λ (Pearce and Wen, 1984). The number of events, M(t), to occur during service life, t, is described by the probability law,

$$P[M(t) = n] = (\lambda t)^n e^{-\lambda t}/n! \; ; \; n = 0, 1, 2, \ldots \tag{4.1}$$

The intensity of each load is a random variable, described by cumulative probability distribution (CDF) $F_l(x)$. If the duration of each load pulse is assumed to be so short that it can be ignored for

practical purposes, the CDF of the maximum load to occur within a given period of time, t, is,

$$F_{max}(x) = \exp[-\lambda t(1 - F_i(x))] \tag{4.2}$$

One can generalize this process to one in which the the load process is intermittent (Figure 4.1b) and the duration of each load pulse has an exponential distribution,

$$F_{T_d}(t) = 1 - \exp[-t/\tau]; \quad t \geq 0 \tag{4.3}$$

in which τ = average duration of the load pulse. The probability that the load process is nonzero at an arbitrary time is $p = \lambda\tau$.

Continuous Load Models. Loads due to normal facility operation or to climatic variations can be modeled by continuous rather than intermittent load processes. A Poisson pulse process, illustrated by the sample function in Figure 4.1c, may be used if the loads are relatively constant for extended periods of time. Changes in the load intensity are assumed to be described by a Poisson process, with rate λ as before; here, however, the duration of each load is exponential, with average duration $\tau = 1/\lambda$. Finally, loads that fluctuate with sufficient rapidity in time that they cannot be modeled by a sequence of pulses can be modeled as Gaussian processes, a sample function of which is shown in Figure 4.1d.

The pulse processes described in Figures 4.1a - 4.1c above can be implemented in the reliability analysis as a finite family of random variables. In contrast, the Gaussian process in Figure 4.1d cannot; other mathematical techniques are required to address such continuous load processes (Melchers, 1987).

Structural loads or actions that may be important for navigation infrastructure include the following (Ellingwood, 1993):

- Dead loads, D
- Operating loads
  - Hydrostatic loads, H_s
  - Hydrodynamic load, H_d
  - Temporal head, H_t
  - Equipment load, Q
- Impact, I_m
- Wind
- Earthquake
- Temperature cycling
- Lateral Soil Pressure

The above loads all can be modeled by one of the stochastic load models summarized in Figure 4.1. Statistical data exist to describe some of these loads (Ellingwood, 1993). Accidental impacts are a particularly significant source of damage to navigation structures, leading to excessive dynamic forces on lock gates as well as abrasion of lock walls that may accelerate damage due to freeze-thaw action. Surveys have been conducted of events that give rise to significant impact (reviewed in Ellingwood, 1993). Most cases of impact arise from lockages, and are caused by unfamiliarity with the lock, irregular flow conditions, and operator error. Impacts occur only rarely in time. Although the Poisson impulse model in Figure 4.1a provides a reasonable stochastic model for impact forces on navigation structures, additional data are required to determine the mean rate of occurrence of impact events and the CDF of intensity. Statistical data for impact, temperature, and lateral soil pressure are unavailable at the present time.

4.2 Statistical Models of Structural Resistance

Previous work to develop probability-based design criteria has focussed mainly on the short-term strength of concrete and steel. Statistical data are available to describe initial strengths of steel and reinforced concrete in the undamaged state (Galambos and Ravindra, 1978; MacGregor, et al, 1983). If the operating environment is benign or the structure is protected, the strength of steel remains constant with time while the strength of the concrete increases. This is illustrated in Figure 4.2, which shows the average strength gain in concrete with age up to 50 years (Washa,et al, 1989). If the concrete is unprotected, this strength increase may be partially or entirely counteracted by time-dependent degradation (Mori and Ellingwood, 1993). Such changes in strength over time must be taken into account in performing a condition assessment of an existing structure or in calculating safety margins or margins to failure.

Structural capacity can be related to the damage penetration, x(t), introduced in Section 3.2, for a given behavioral limit state of interest. For example, for a reinforced concrete wall loaded in flexure, the time-dependent flexural strength due to corrosion of reinforcement would be,

$$M(t) = A_s(t)\ f_y\ j(t)d \tag{4.4}$$

in which f_y = yield strength of reinforcement, $j(t)d$ = internal moment arm, and $A_s(t)$ = time-dependent area of reinforcement, given in terms of x(t), initial bar area, A_s, and bar diameter, d_b, as,

$$A_s(t) \approx A_s\ (1 - 4x(t)/d_b) \tag{4.5}$$

the approximation in Eqn 4.5 holding for small amounts of corrosion. Similar expressions can be derived for other strength limit states. Statistical characteristics of the degradation term x(t) remain to be determined.

The structural resistance for a particular limit state of interest (strength or serviceability) is described by a cumulative probability distribution function (CDF). In a hostile environment and in the absence of any corrective action, the CDF will shift toward smaller values of strength with time as the structural materials suffer environmental attack, as shown in Figure 4.3. In-service inspection and repair can retard or reverse this process, depending on its effectiveness. In some instances, unnecessary repairs or repairs by unqualified personnel may actually accelerate the degradation; this is not uncommon when welded structures are repaired, but may occur in other instances as well. The role of in-service inspection/repair in reliability maintenance is considered in Section 5.

4.3 Reliability Analysis of Degrading Structures

To illustrate the reliability analysis of a degrading component in simple way, let us assume that significant structural loads can be modeled as a sequences of pulses, the occurrence of which is described by a Poisson process with mean rate of occurrence, λ, random intensity, S_j, and duration τ. Concurrently, the strength of the structure, R(t), decreases over time due to environmental attack. Sample functions of load and strength are illustrated in Fig. 4.3. The limit state of the structure at any time, t, can be described by the function,

$$R(t) - S(t) < 0 \tag{4.6}$$

and the probability of failure at time, t, is $P[R(t) < S(t)]$. The reliability function, L(t), is defined as the probability that the structure survives during interval of time (0,t). Taking into account the randomness in the number of loads and times at which they occur, L(t) is (Ellingwood and Mori, 1993),

$$L(t) = \int_0^\infty \exp\left[-\lambda t\left[1-\frac{1}{t}\int_0^t F_S(g(t)r)dt\right]\right]f_R(r)\,dr \tag{4.7}$$

in which $f_R(r)$ = probability density function (PDF) of initial strength, R, and $g(t) = R(t)/R(0)$ describes the time-dependent degradation in strength described, e.g., by Eqn 4.4 for flexure. The reliability function and limit state probability, $P_f(t)$, are related by $L(t) = 1 - P_f(t)$.

The probability of survival during interval (0,t) can be expressed in terms of the conditional failure rate or hazard function, h(t), defined as,

$$h(t) = -\frac{d}{dt} \ln L(t) \tag{4.8}$$

Integrating Eqn 4.8 to obtain L(t), we obtain

$$L(t) = \exp\left[-\int_0^t h(\xi)d\xi\right] \tag{4.9}$$

If the structure has survived during interval $(0,t_1)$, it may be of interest to determine the probability that it will fail before t_2. Such a probability may be used, for example, in scheduling some future inspection and maintenance action at future time t_2, based on what has been learned at t_1. This probability can be expressed in terms of the hazard function:

$$P_f(t_2|T_f>t_1) = 1 - \exp\left[-\int_{t_1}^{t_2} h(\xi)d\xi\right] \tag{4.10}$$

When structural failure occurs due to aging or deterioration, h(t) increases with time. The effect of strength degradation on the reliability and hazard functions for a structure are illustrated conceptually in Figure 4.4.

The reliability and limit state probability functions L(t) and $P_f(t)$ are cumulative, that is, they describe the probabilities of successful (or unsuccessful) performance during a service interval (0,t). It should be emphasized that $P_f(t) = 1 - L(t)$ is not equivalent to $P[R(t) < S(t)]$; the latter probability is simply the probability of failure at time, t, without regard to previous or future structural performance. Failure to recognize the difference between these probabilities is a fundamental but common interpretive error.

As a practical illustration of these concepts, we consider a reinforced concrete slab that has been designed to the requirements of ACI 318-89 (Ellingwood and Mori, 1993). The slab is subjected to a stochastic load in which the significant events occur once every two years, on average; the mean transient load intensity is 0.4 time the nominal design load and the coefficient of variation (COV) in intensity is 0.5. The mean moment capacity is 1.15 times the nominal ACI moment capacity, and the COV in strength is 0.15. The degradation process is assumed to be such that 90% of the initial design strength remains at 40 years in all cases; however, α in Eqn 3.4 is 1/2 (square root), 1 (linear) and 2 (parabolic). The results of the reliability analysis are shown in Figure 4.5. Note that aging processes, if uncorrected, may raise the limit state probability during a service life of 60 years by as much as an order of magnitude.

It should be remarked that little research has been published to support probabilistic analysis of structural degradation due to corrosion (Komp, 1987; Rogers, 1990), stresses due to shrinkage or creep (Bazant, 1986), freeze-thaw (Bryant and Mlakar, 1990), or expansive aggregate reactions (Mori and Ellingwood, 1993). Additional studies are required to implement such analyses as a basis for risk management of aging navigation structures.

If degradation occurs due to fatigue damage accumulation under variable amplitude loading during interval (0,t), we have from Eqn. 3.10 that:

$$D(t) = \sum_{i=1}^{M(t)} C^{-1} S_i^m \tag{4.11}$$

Failure occurs when $D(t) > \Delta$, in which Δ is a random variable that accounts for uncertainty in Miner's rule at failure. Parameter Δ often is assumed to be lognormal, with median $M_\Delta = 1.0$ and $SD(\ln \Delta) = 0.30 - 0.60$ (Committee, 1981; Yao, et al, 1986; Torng and Wirsching, 1991). If the damage increments are small, M(t) is large and the stress process is stationary and narrow band, the expected value of D(t) is,

$$\begin{aligned} E[D(t)] &= E[M(t)] \;\; E[C^{-1} \; S^m] \\ &= C^{-1} \, (\nu t) \;\; E[S^m] \end{aligned} \tag{4.12}$$

in which $E[M(t)] = \nu t$, ν = mean cycling rate, and $E[S^m]$ is the m_{th} moment of ΔS, determined from

$$E[S^m] = \int_0^\infty s^m f_S \; (s) ds \tag{4.13}$$

in which $f_S(s)$ = probability density function of stress range, determined from the operating load history. The variance of D(t) is difficult to obtain; in general, $Var[D(t)] \sim 1/\sqrt{t}$. Thus, assuming that t is large, the fatigue life can be defined at the point at which $E[D(t)] = 1.0$ (Lutes, et al, 1984). This assumption leads to the relation (in which $N_t = \nu t$):

$$N_t \, E[S^m] = C \tag{4.14}$$

A similar formulation can be obtained from the crack growth equation,

$$da/dN = C(Y \; S\sqrt{\pi a})^m \tag{4.15}$$

Assuming that the loading can be modeled by a sequence of random loads, S_i, we have (Casciati and Colombi, 1993),

$$\int_{a_o}^{a_t} (Y\sqrt{\pi a})^{-m}\, da = C \sum_{i=1}^{M(t)} S_i^m \tag{4.16a}$$

It can be shown that

$$\lim_{t \to \infty} C \sum_{i=1}^{M(t)} S_i^m = (\nu t)\ C\ E[S^m] \tag{4.16b}$$

if the stress process is stationary and narrow-band. Note the similarities of the right hand side of Eqn 4.16b with 4.12. If the statistics of initial and final defect sizes, a_o and a_t, are known, the probability of unacceptable defect growth (and, thus, when failure might occur or repair might be required) can be obtained from Eqns 4.16 (e.g., Oswald and Schueller, 1984; Ortiz and Kiremidjian, 1988).

For the fatigue limit state, the limit state probability can also be evaluated as,

$$P(N_t < N_{req'd}) \tag{4.17}$$

in which $N_{req'd}$ = required service life, expressed in load cycles. The hazard function for fatigue is described by an increasing failure rate model (cf Figure 4.4), often assumed to be

$$h(t) = \left(\frac{\alpha}{u}\right) \left(\frac{t}{u}\right)^{\alpha-1} \tag{4.18}$$

in which α and u are parameters and the number of cycles and time are related by $N_t = \nu t$. The CDF for cycles to failure, N_t, derived from this model is,

$$F_T(t) = 1 - \exp\left[-\left(\frac{t}{u}\right)^{\alpha}\right];\ t \geq 0 \tag{4.19}$$

sometimes referred to as a Weibull distribution. The coefficient of variation in fatigue life typically is 0.4 or greater (Committee, 1981).

The main factor differentiating in a reliability sense an aging or deteriorating structure from one that is protected is the hazard function or failure rate function (Figures 4.4 and 4.5a). The

increase in h(t) (the so-called increasing failure rate or IFR function) is characteristic for aging structures; for random or purely chance failures, h(t) is constant. From the point of view of time-dependent reliability, in-service inspection and maintenance changes h(t). We will return to this subject in Section 5.

4.4 Reliability of Systems

The analysis of the reliability of a system of structural components or a system of facilities is conceptually similar to the analysis of a single component. However, the multiplicity of possible failure modes in a complex system makes the analysis considerably more complex mathematically.

4.4.1 Basic Concepts

The limit state probability of a system can be formulated in terms of the limit state probabilities of the individual components in the system or limit state probabilities of individual modes, depending on how the system is modeled mathematically (Moses, 1990; Karamchandani, et al, 1994).

At the most fundamental level, there are two basic types of systems: series and parallel systems. In a series system, failure occurs if any component (mode) fails. In a three-component system, for example,

$$F_{sys} = F_1 + F_2 + F_3 \tag{4.20}$$

in which F_{sys} = failure event for the system, F_i = failure event for component i, and the symbol "+" denotes union. In a strictly parallel system, failure occurs only if all components (modes) fail. For the three-component system above,

$$F_{sys} = F_1 * F_2 * F_3 \tag{4.21}$$

in which the symbol * denotes intersection. Most realistic engineered systems must be modeled as combinations of unions and intersections; for example,

$$F_{sys} = F_1 + (F_2 * F_3) \tag{4.22}$$

meaning that the system fails either if component 1 fails or if 2 and 3 both fail. System limit state probabilities are difficult to compute, except in special cases. There is, first of all, the issue of modeling the individual component (mode) failure events in terms of the basic random variables. The

impact of stochastic dependence among failure events also must be considered. Finally, the computation of the joint probability integrals is numerically difficult.

4.4.2 Reliability analysis by response surfaces

Modern structural analysis often is performed using finite element methods. Finite element analysis (FEA) is algorithmic in nature, yielding structural responses of interest at discrete points, but not a general closed-form expression for the limit state function, such as in the simple illustration in Section 4.3. One can, however, use FEA to develop a "response surface," an approximation to the limit state surface that is sufficiently accurate to be used in reliability analysis (Bucher and Borgund, 1990; Rajashekhar and Ellingwood, 1993). First, the FEA is performed at a set of carefully pre-selected experimental points in the domain of random variables. Next, a response surface model is fitted to the finite element results by regression or interpolation analysis. Response surfaces commonly are assumed to be second-order polynomials. Suppose that the exact margin of safety is denoted,

$$z = g(X_1, X_2, \ldots, X_n) \tag{4.23}$$

in which $g(\ )$ is a (explicit or implicit) function describing structural behavior and X_i, $i = 1,\ldots,n$ are random resistance and load variables. The limit state is defined by the condition $z < 0$. If $g(\ )$ is an implicit function determined algorithmically by FEA, we might replace the exact value by,

$$\hat{z} = a_o + \sum_{i}^{n} b_i x_i + \sum_{i}^{n} \sum_{j}^{n} c_{ij} x_i x_j \tag{4.24}$$

The constants a_o, b_i, c_{ij} are determined so as to minimize

$$I(a_o b_i c_{ij}) = \sum_{k} (z_K - \hat{z}_K)^2 \tag{4.25}$$

in which z_k is obtained from FEA at the kth experimental point $x_k = (x_1, x_2, \ldots, x_n)_k$ and $\hat{z}_k$ is determined by Eqn.4.24 at the same point. Once an adequate representation of z has been obtained, the limit state probability can be determined by first-order reliability analysis, importance sampling or other numerical techniques (e.g., Schueller and Stix, 1987; Melchers, 1989).

Any response surface has associated statistical error. Response surfaces created by interpolation are exact at the experimental points, but approximate between them. Surfaces determined by regression produce a fit that is best in an overall (mean-square) sense, but in error practically everywhere. Response surfaces created by numerical experiments (e.g., by FEA) contain

statistical errors due to fitting, omission of key variable(s), or the use of small samples (residual error). For a given number of experiments, increasing the number of constants to be determined decreases the fitting error but increases the error in the residual. The statistical error created by the response surface process should be included in the reliability analysis. Experiments for determining response surfaces must be planned carefully; factorial and central composite designs both have been used (Faravelli, 1989; Bucher and Bourgund, 1990; Engelund and Rackwitz, 1992; Rajashekhar and Ellingwood, 1993). Techniques also have been proposed for improving the response surface for reliability analysis purposes by successive iteration.

The response surface technique appears promising as a method to evaluate the probability of damage due to freezing and thawing in navigation structures. The probability of freeze-thaw damage at time, t, and at point (x,y) in a concrete wall with water on either side (see Figure 4.6a) can be expressed as (Bryant and Mlakar, 1990),

$$P_d(t) = P[T \leq T_{cr} * S > S_{cr}] \tag{4.26}$$

in which $T(x,y,t)$ = temperature, $S(x,y,t)$ = percent saturation, and T_{cr} and S_{cr} = critical values of temperature and saturation for the expansive phase change to occur, and symbol * denotes intersection of the events. This probability was estimated by assuming that: (1) the events $(T < T_{cr})$ and $(S > S_{cr})$ are statistically independent; (2) $T(x,y,t)$ can be obtained by solving a one-dimensional heat flow equation; (3) the concrete is fully saturated below the phreatic surface, enabling $P(S > S_{cr})$ to be determined simply from the statistics of pool elevation on either side of the wall; and (4) statistics of T and S can be obtained by approximate second-moment analysis of T and S. Assuming that T and S can be modeled as normal random variables, equi-probability contours of $P_d(t)$ were constructed, as illustrated in Figure 4.6a; contours of useable life were determined as, $L_d(t) = 1/P_d(t)$.

The proposed approach is relatively simple. However, several potential shortcomings can be identified: (1) because pool elevations change continuously in time, the concrete below the phreatic surface at any given time may not be fully saturated; (2) the use of a one-dimensional thermal analysis to determine temperature distribution may be not be sufficiently accurate; (3) the event that $(T \leq T_{cr} * S > S_{cr})$ at a point (x,y) may not imply that the concrete is no longer able to carry load at that point, particularly if the concrete is confined; (4) the equi-probability contours of P_d are not related to the degraded structural capacity required for a time-dependent reliability analysis.

The determination of the internal temperature distribution, moisture ingress and saturation can be determined by finite element analysis. The evaluation of saturation presents a particularly difficult challenge. Moisture ingress occurs by the mechanisms of seepage (convection-related, dependent on head and permeability of concrete) and by capillary action. By an appropriate selection of experiments, it should be possible to construct a response surface describing the internal state of the wall in terms of freeze-thaw damage. Such a response surface could be used to determine contours of damage and as an independent check on the validity of simpler, closed-form approximations.

Pointwise estimates of damage probability cannot be used directly in time-dependent reliability assessment. The damage probability contours must be related to structural capacity. As

shown in Figure 4.6b, damage probability can be related to damage volume by a curve that resembles a complementary cumulative probability distribution function (CCDF). It is known from probability theory that if a random variable, X, is distributed on the interval (0,W), its mean and variance can be determined by,

$$E[X] = \int_0^W [1 - F_X(x)]dx \tag{4.27}$$

$$Var[X] = \int_0^W 2x[1-F_X(x)]dx - (E[X])^2 \tag{4.28}$$

Substituting the CCDF in Figure 4.6b for $[1 - F_X(x)]$ in Eqns 4.27 and 4.28, the mean and variance of damage area or volume can be determined from the contours of damage probability. These statistics can be used subsequently to evaluate the statistics of strength degradation with time.

The determination of the damage contours in Figure 4.6 is key to being able to assess the impact of freeze-thaw damage on time-dependent reliability. Additional research is required to develop and implement the finite element analysis leading to the response surfaces that are needed to perform this assessment.

4.5 Structural Damage Modeling

In a general sense, the state of a structural system at any given time can be expressed by the "damage" that it has incurred during its service life to that time. A damage analysis can be performed to determine the degree to which the structural system has suffered a loss of capacity to withstand operating loads or loads due to natural hazards. Several methods and quantitative measures have been proposed to model damage (e.g., Whitman, et al, 1975; Park, et al, 1985; Stephens and Yao, 1987). Fragility modeling is particularly attractive as a tool for probability-based condition assessment.

4.5.1 Damage modeled as a Markov chain

Damage accumulation in a structure is a time-dependent process. The progression of damage in a structure can be modeled as a Markov chain (Rahman and Grigoriu, 1993; Bogdanoff and Kozin, 1985). The Markov model does not provide any insight in the mechanics of the degradation process. It simply provides a convenient data management system for handling the evolution of damage state probabilities over time and, by its matrix formulation, provides a algorithm that lends itself well to computerization.

In its simplest implementation, one might envision the structure to be in one of i = 1,...,n discrete "states" of damage, D, any given time, t, in which 1 may correspond to "undamaged" while "n" corresponds to "failed." The probability that the structure is in damage state, i, at time t is,

$$P(D(t) = i) \;=\; p_i(t),\; i = 1,2,...,n \tag{4.29}$$

At some later time $t + \Delta t$, the probability that the structure is in state j is,

$$P[D(t+\Delta t) = j] = \sum_{i=1}^{j} P[D(t+\Delta t)=j|D(t) = i] \quad P[D(t) = i] \tag{4.30}$$

in which $P[D(t+\Delta t) = j|D(t)= i]$ = probability that the structure is in state j at time $t + \Delta t$, given that it was in state i at time t. Eqn 4.30 is simply a statement of the theorem of total probability. Considering all 1,...,n damage states, Eqn 4.30 can be expressed in matrix form as,

$$P(t+\Delta t) = T\, P(t) \tag{4.31}$$

in which T is denoted the "transition probability matrix."

The nature of the degradation process determines the characteristics of transition probability matrix, T. With the observation that the structure cannot heal or repair itself in the absence of some outside agent, T is a lower triangular matrix if the vectors $P(t+\Delta t)$ and $P(t)$ are expressed in column form. If damage growth is gradual or Δt is small, only small changes of state are possible during Δt. In such cases, T would be nearly diagonal. Finally, if the degradation process is stationary in the weak sense, meaning that the first-order statistics do not change in time and the joint second-order statistics are dependent only on Δt, then T depends only on the existing damage at t and is simply a function of that damage and Δt. Such an assumption requires that extrinsic factors are not influential in the rate of damage growth.

As an illustration of this concept, suppose that the states of a hydraulic structure in which damage is accumulating can be grouped (for simplicity) into four categories: (1) undamaged; (2) lightly damaged; (3) moderately damaged; and (4) severely damaged. The state probability vector at time $t = n\Delta t$ is,

$$P(n\Delta t) = \prod_{i=1}^{n} T_i\,(\Delta t)\;\; P(0) \tag{4.32}$$

in which $P(0)$ = initial state probability vector. The time is discretized into ten-year intervals and the damage process is assumed to be stationary in the weak sense. Let us assume that the transition probability matrix for each ten-year interval is,

$$T = \begin{bmatrix} 0.95 & 0.0 & 0.0 & 0.0 \\ 0.05 & 0.90 & 0.0 & 0.0 \\ 0.0 & 0.10 & 0.85 & 0.0 \\ 0.0 & 0.0 & 0.15 & 1.0 \end{bmatrix} \tag{4.33}$$

and that the initial state vector (at $t = 0$) is $P(0) = (0.98,0.02,0,0)^t$. The damage state vector at 40 years can be obtained from Eqn 4.32 as $P(40) = (0.798,0.324,0.053,0.007)^t$.

To perform such an analysis, damage must be related to a measurable physical quantity. The elements of T can be determined by an expression similar to Eqn 4.10 or by the use of Eqns 4.12 or 4.16 with an appropriate service load spectrum. However, the exact means by which this is done is beyond the scope of this illustration and requires further research.

4.5.2 Fragility modeling concepts

Fragility is a concept that was introduced to represent probabilistically the capacity of a system to withstand some extreme demand in a simple way. It can be determined in a variety of ways, ranging from simple heuristics to complex stochastic finite element analysis. The fragility concept has found widespread usage in the nuclear power area, where it is used in performing seismic probabilistic safety and/or margin assessments of safety-related systems (Kennedy and Ravindra, 1984). Its use as a basis for decision-making regarding Corps facilities has not been explored and warrants further investigation because of its power and simplicity.

The fragility of a component is defined as the probability that component reaches some limit state, LS, conditioned on the occurrence of a particular value, y, of some random demand, Y. In equation form,

$$\text{Fragility} = P[LS|Y{=}y, \theta] = F_R(y) \tag{4.34}$$

in which $P[LS|Y{=}y,\theta]$ = conditional probability of LS, given y, and θ = vector of fragility parameters. The demand, y, might be expressed as an effective peak ground or spectral acceleration if the demand is due to earthquake, a windspeed if due to wind, or a static head if due to flood. The fragility parameters might include the material strength statistics, structural dimensions, dynamic properties of the system, and so forth.

The first step in generating fragilities is to develop a clear definition of what constitutes the performance limit states in critical systems within the facility. This introspective activity on the part of facility operators and engineers is very useful as it focusses their attention on what can go wrong with a facility and how to mitigate such events. Such thought processes seldom occur at the facility design stage. The limit states can range from functional failures, such as equipment malfunction, all the way to large inelastic deformations or structural collapse. Numerous sources of uncertainty in the prediction of the fragility also should be considered; these include inherent randomness in the material strengths and dimensions, dynamic characteristics, and the ability of the

fragility (structural) analyst to model the systems accurately. Additional uncertainties arise from limitations in the supporting databases.

In seismic fragility analysis, it has been common to posit a lognormal model for the fragility (Kennedy and Ravindra, 1984):

$$F_R(y) = \Phi\left(\frac{\ln(y/M_R)}{SD(\ln R)}\right) \tag{4.35}$$

in which $\Phi(\ )$ = standard normal CDF, M_R = median (50th percentile) fragility, and SD(lnR) = logarithmic standard deviation that is approximately equal to the COV in R, describing the inherent variability in the capacity of the component to withstand acceleration of intensity x.

The fragility of a system of components (structural system) can be determined in exactly the same way if a means to formulate the system reliability in terms of the component (or modal) reliabilities is available. For example, if the failure of a system of three components can be expressed as in Eqn 4.22 and the three component failure events are statistically independent, then the fragility of the system is simply,

$$F_{R_{sys}}(y|\theta_1\theta_2\theta_3) = F_{R_1}(y|\theta_1) + [\ 1-F_{R_1}(y|\theta_1)]\ \ F_{R_2}(y|\theta_2)\ F_{R_3}(y|\theta_3) \tag{4.36}$$

Uncertainties in modeling each component fragility are vested in its estimated median, M_R, which also is assumed to be a lognormal random variable with median m_R and logarithmic standard deviation SD(lnM_R). The fragility for a component or system thus can be described by a family of lognormal distributions, as illustrated in Figure 4.7.

The system fragility provides a tool for assessing the capacity of a facility when there is limited or no probabilistic information to describe the hazard. As an example, suppose that it is desired to assess whether a system with fragility described in Figure 4.7 is likely to survive an earthquake causing a effective peak ground acceleration of 0.2g at the facility site. No other information on the earthquake hazard is available; presumably, seismologists have informed the facility manager that capable seismogenic sources in the region are unlikely to generate earthquakes larger than 0.2g. (In the nuclear industry, such an earthquake is referred to as a "review" or "screening level" earthquake.) For the system described by the fragility family illustrated in Figure 4.7, the mean of the estimated 5% exclusion limit of capacity (measured in units of ground acceleration) is approx. 0.6g; the lower 95% confidence interval on that estimate is 0.34g. Thus, the facility has a capability (with 95% confidence) to withstand an earthquake with ground acceleration 1.7 times greater than 0.2g with 95% reliability. Such a statement gives some sense of the level of conservatism to which the facility may be expected to operate. (In seismic margins analyses conducted for nuclear power plants, such a value is referred to as a "high-confidence, low probability of failure" or HCLPF capacity.) Using this notion, one can perform a probabilistic assessment of safety margins with respect to design-basis events that may or may not have a statistical rationale.

The hazard for the facility from an environmental event can be described by a curve, G(y), displaying the probability that a given intensity, y, of that event is exceeded in a given period of time, often taken as one year. If G(y) for the facility is available and the limit state probability of a component or system is required for decision analysis, that probability can determined by convolving the derivative of the fragility with the hazard curve. The (unconditional) limit state probability becomes,

$$P[LS|\theta] = \int_0^\infty G(y) \frac{d}{dy} F_R(y)dy \tag{4.37}$$

Note that this probability is a function of the fragility parameters assumed; other assumptions will lead to other limit probability estimates. If the parameters, θ, are also assumed to be described by probability distributions, one obtains a frequency distribution of limit state probability. A hazard curve requires a site-specific analysis of the facility for the operating or environmental hazards of interest. The technology to perform site-specific analyses has advanced considerably during the past decade (e.g., Reiter, 1990) but remains controversial in certain applications. Moreover, an acceptable risk must be available against which Eqn 4.37 can be compared. The difficulties in defining an appropriate measure of risk are discussed in Section 6.

One of the attractive features of fragility analysis is that it can be performed at several levels, consistent with the level of data and sophistication of the reliability analysis techniques available. Current methods for assessing component or system fragilities are based largely on an extrapolation of design data to model the in-situ structure, a process that relies to a considerable degree on subjective judgement. While the process may contain large uncertainties, these uncertainties can be reduced with the use of more sophisticated analytical tools. Frequently, this is accomplished with the assistance of an expert panel, with considerable interchange among the members in establishing the fragility parameters. The decision process leading to the fragility model is transparent and scrutable, and involves both the engineer and facility operators in a mutually reinforcing way. It provides an audit trail for decisions regarding safety and function of components and systems. Moreover, it provides a tool to ensure that whatever information on uncertainties is available is treated in a consistently. The role of fragility modeling in performance assessment of navigation structures requires further investigation.

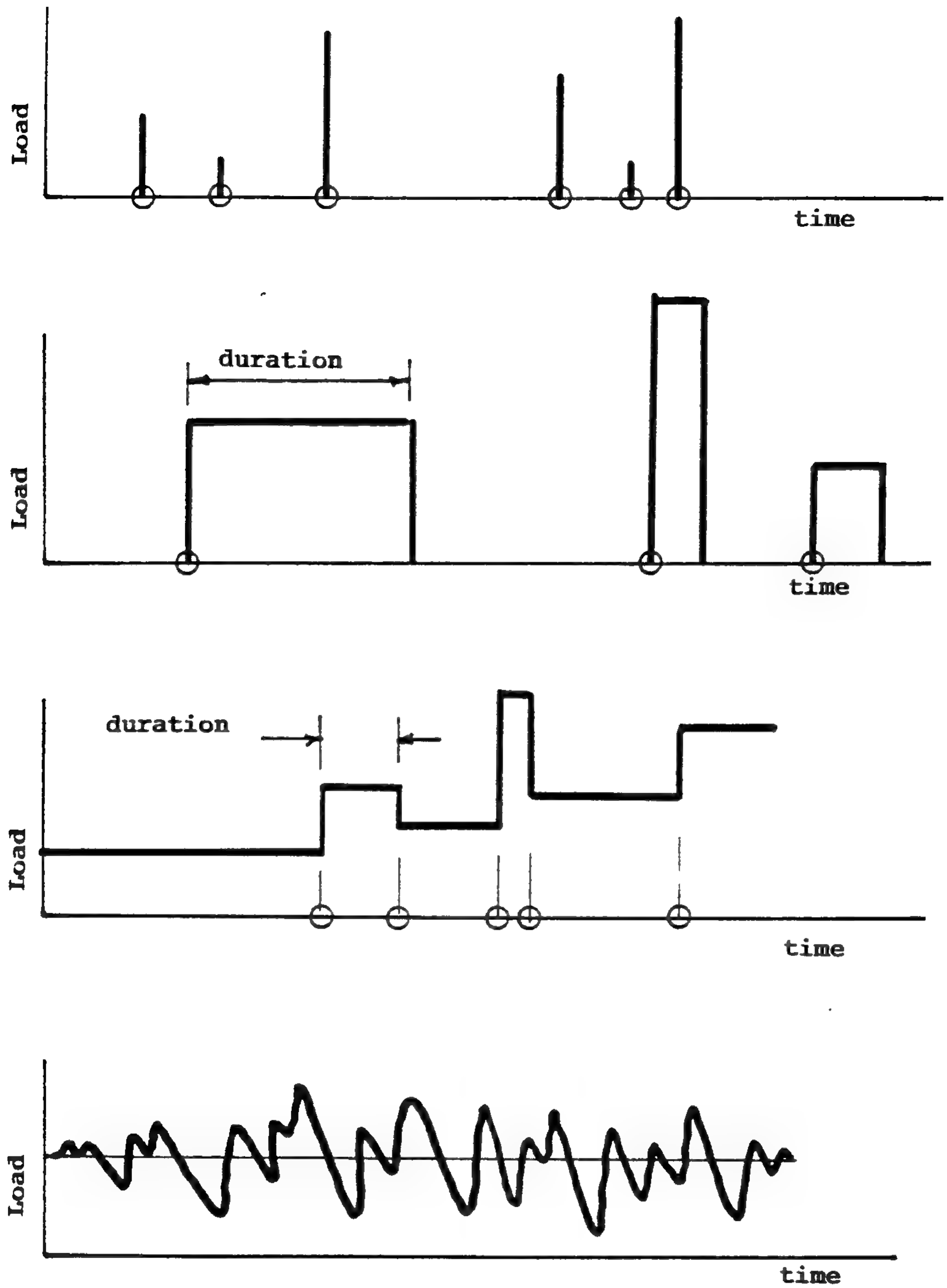


Figure 4.1 - Stochastic load models

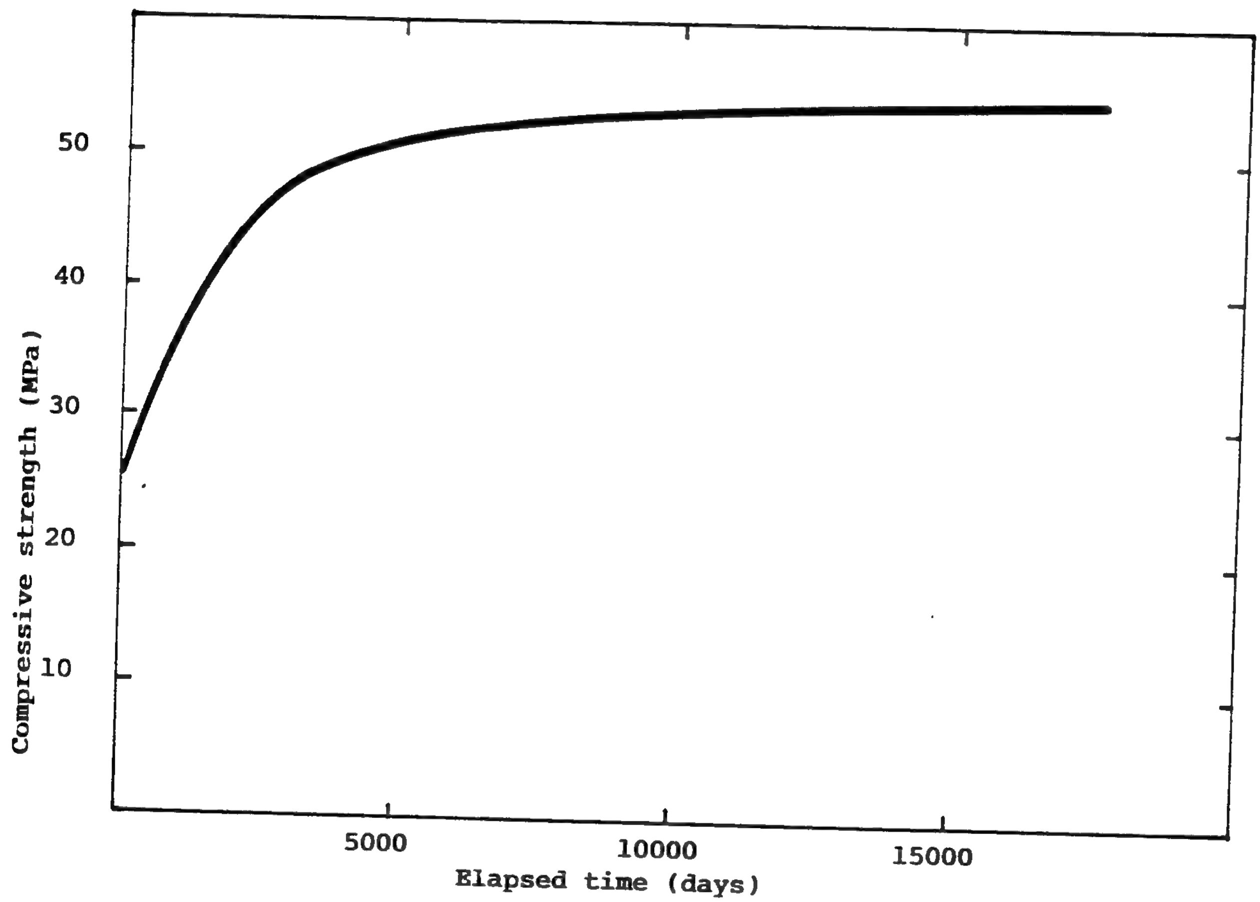


Figure 4.2 - Compressive strength of concrete vs time (Washa, et al, 1989)

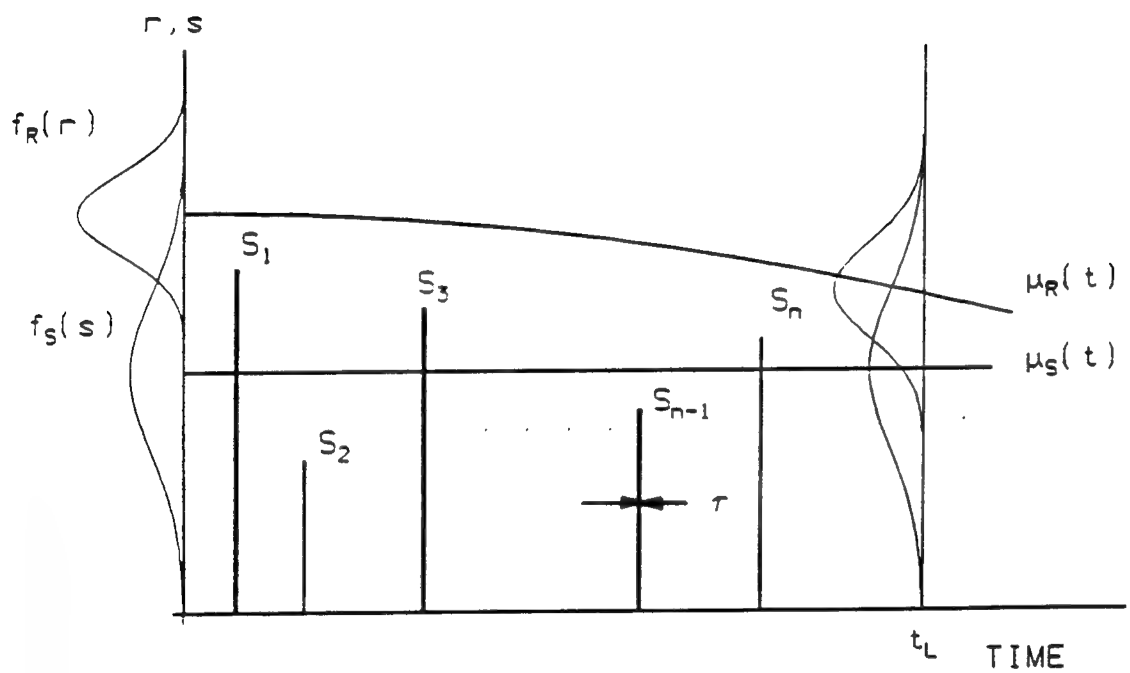


Figure 4.3 - Sample functions representing structural load process and degradation of resistance

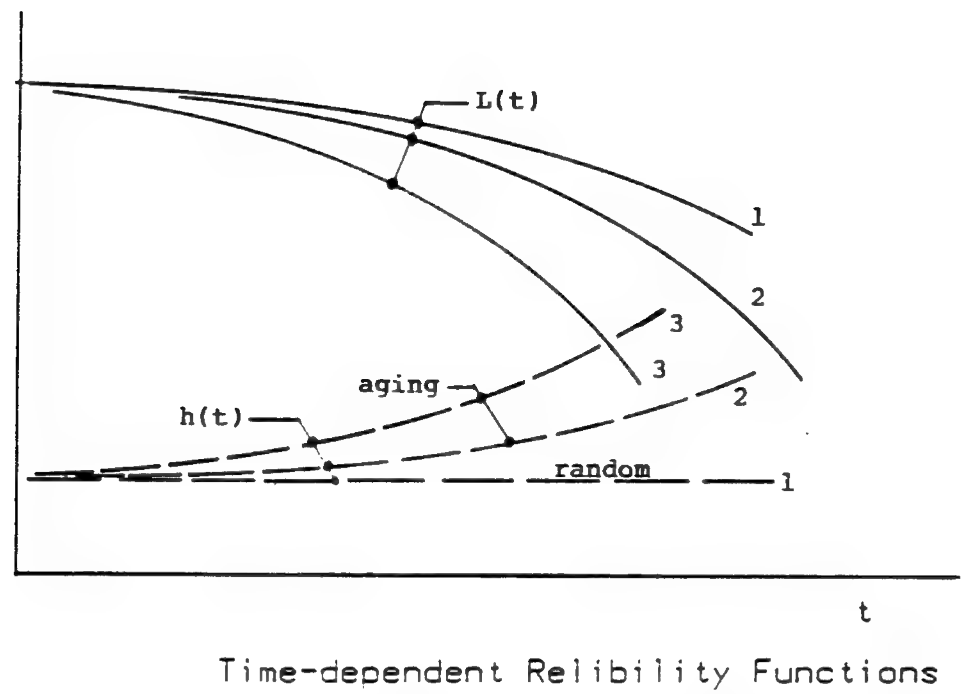
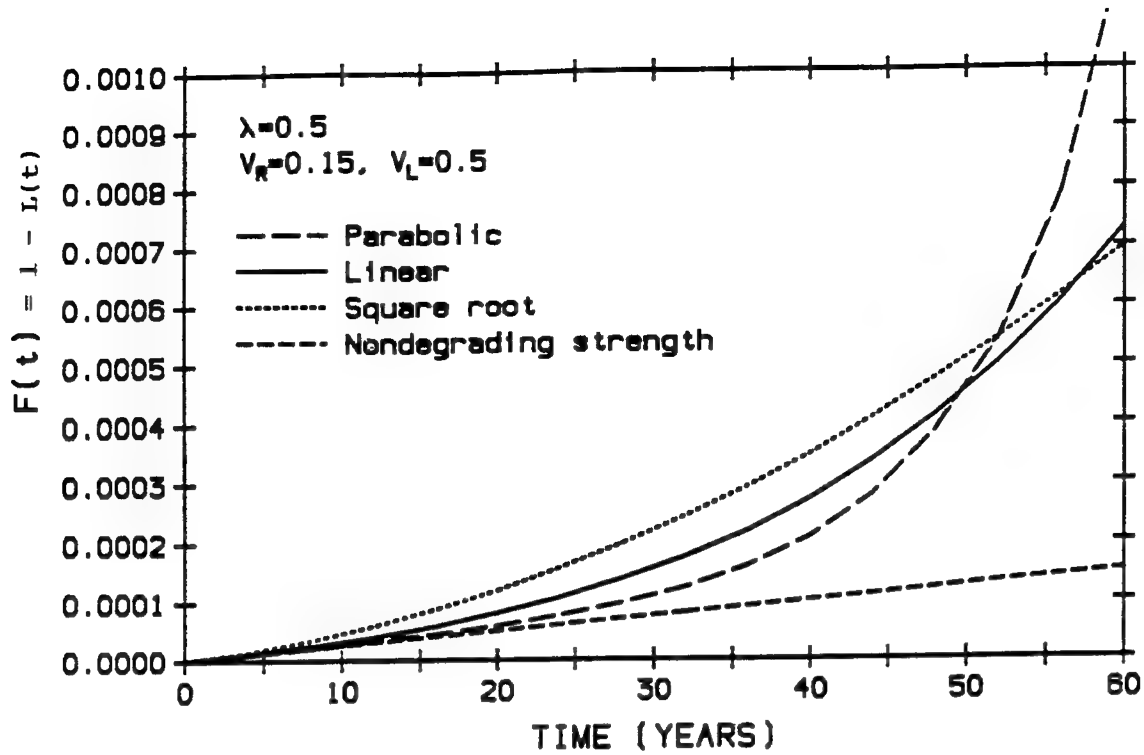


Figure 4.4 - Time-dependent reliability and hazard functions



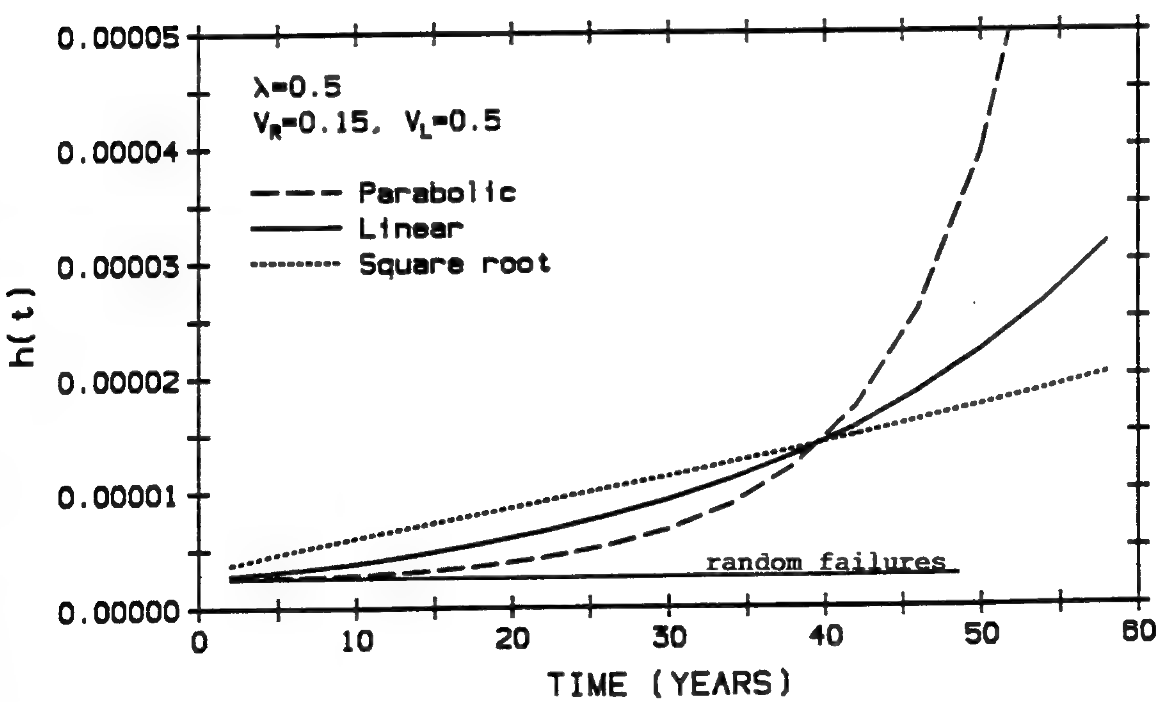
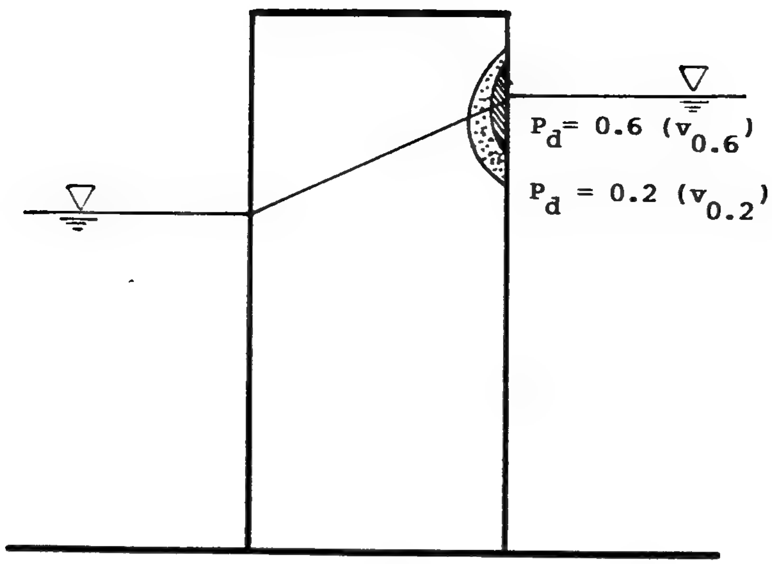


Figure 4.5 - Illustration of time-dependent reliability analysis (Ellingwood, and Mori, 1993)



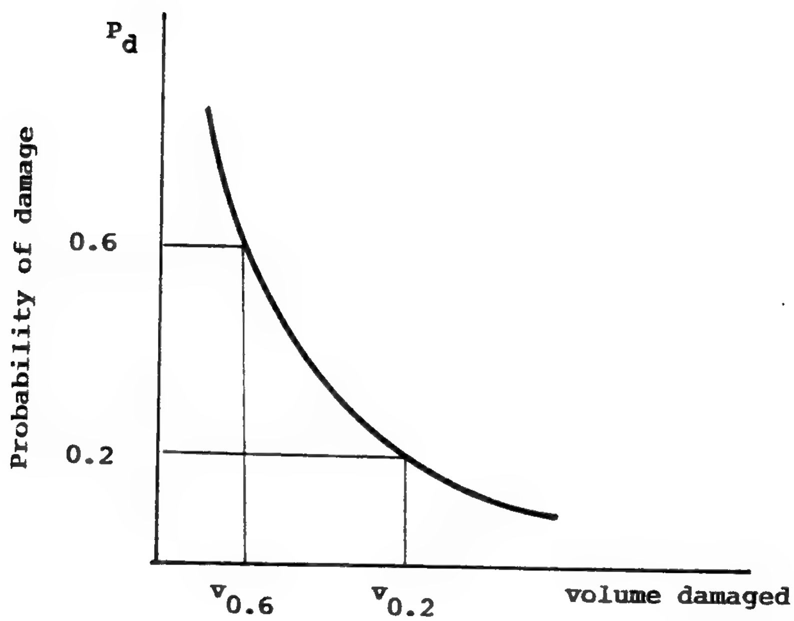


Figure 4.6 - Concrete wall subjected to freeze-thaw damage

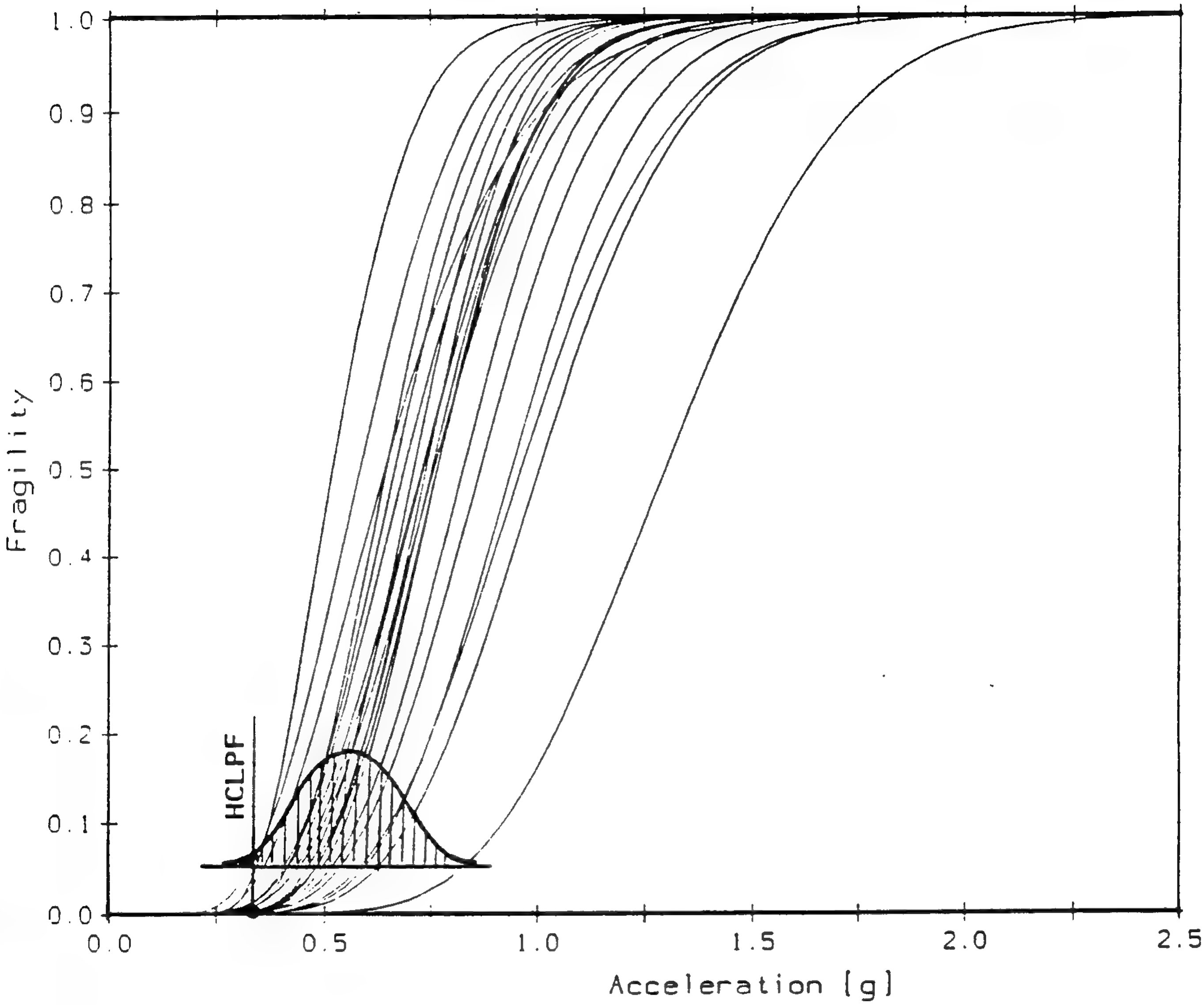


Figure 4.7 - Illustration of seismic fragility family

5. ROLE OF IN-SERVICE INSPECTION AND MAINTENANCE IN RISK MANAGEMENT

5.1 Overview of in-service inspection methods

In-service inspection and maintenance (ISI/M) provides a means for minimizing the impact of aging on structural performance. The aims are to identify the cause and extent of damage, evaluate residual strength and serviceability, and to provide recommendations on remedial or preventive measures (Chung, 1994). Efficient and accurate nondestructive evaluation (NDE) methods, particularly those that are noninvasive and do not disrupt the use of the structure, are essential for a properly designed ISI/M program.

At present, most facility operators do not conduct structural inspections regularly. Those that do inspect, do so at irregular intervals and rely almost exclusively on visual inspection. In-service inspections tend to be reactive, occurring only after there is some visible indication of damage or performance has been impaired. Operating budgets frequently do not provide for routine inspection. This lack of foresight and attitude toward ISI/M must change if life-cycle risk management is to be implemented.

A detailed condition survey is necessary at the initial stages of a inspection/repair operation to identify the scope of the problem and environmental exposure (Krauss, 1993). An authoritative document for performing such a survey is ACI 201 (Guide, 1989), which also contains a inspection checklist. Design documentation and records of construction and repairs and service history should be reviewed at this time. The initial inspection, often performed visually, can document information on cracking, spalling, leakage, evidence of chemical attack, and other factors that may lead to structural deterioration, and indicates where to concentrate further quantitative testing procedures.

In-service inspection should be oriented toward quantifying defect parameters that relate to and can be incorporated in calculations of the degraded strngth of a structural component. Many of the standard nondestructive evaluation (NDE) techniques were developed for the detection and assessment of defects arising from manufacturing. Such techniques may not be effective or adequate for in in-service NDE of aging structures. Measurement conditions in-service often are poor, operators are not as well trained, and structural components may be inaccessible or inconvenient to inspect. The most effective NDE usually involves some combination of testing procedures. In-situ structural strength varies over a structure or even within a large structural component, depending on the environmental conditions to which it is exposed. Accordingly, repetitive sampling is necessary to obtain reliable estimates of in-situ strength. An appropriate repetitive sampling plan must be established, in terms of both combining NDE procedures and determining the number of samples from each method, to maintain the desired level of reliability and performance.

Visual inspection is most common, often accompanying routine maintenance, and allowing easy identification of cracking, spalling, surface defects, and potential sites of corrosion. Visual inspection is least susceptible to operator error, requires no special equipment, and often is highly effective in locating preliminary indications of structural damage, provided that the inspector is experienced and looks in the right place. However, little information on the interior condition of the component or on the strength is obtained without resort to other methods. For some limit states such as fatigue or internal deterioration, visual inspection provides little useful information.

Corrosion of reinforcement near the surface of a concrete structure often can be detected by visual examination, since expansive corrosion products in the outer layers is most likely to cause cracking, spalling and staining. Half-cell potential measurements can be used to detect (with specified probability) the occurrence of corrosion zones in reinforcement within the concrete (ASTM C876-91). However, the technique does not give any information on the degree of damage, only its presence.

More sophisticated NDE methods involve measurement of propagation characteristics of stress or electromatic waves. Flaws or discontinuities in a solid disrupt the wave propagation patterns. Spectral analysis of wave patterns can be used to identify locations and sizes of voids or other defects. Ultrasonic pulse velocity and impact-echo testing are two such methods that have been widely used in concrete (Malhotra and Carino, 1991). Correlations between pulse velocity and concrete strength also can be developed; such correlations are sensitive to the composition of the concrete mix and must be developed for a specific mix before being used to determine in-situ strength. Some of these methods require access to both sides of the component; others to only one side. Ultrasonic and radiographic techniques can be highly effective in locating subsurface defects, but are more difficult to use in the field and require highly trained operators. They are commonly used in metallic structures; the applicability and advantages and disadvantages of these methods are well known (Barsom and Rolfe, 1987).

In-situ strength of concrete can be measured indirectly by rebound or probe tests, using correlations between the test measurement (height of hammer rebound or penetration) to compressive strength. Such methods are sensitive to the surface hardness and aggregate as well as the mass of the component tested. Drilled cores from concrete components yield more reliable estimates of in-situ strength of concrete than indirect estimates obtained from the methods described above (Bungey, 1989). Unfortunately, drilling cores is semi-destructive in nature. Moreover, since strength generally varies over a structure, one needs to take a number of randomly selected samples in order to obtain a representative measure of strength. Tensile coupons from steel components can be used to estimate their strength; however, steel coupon testing is not common unless it is believed that the steel has been accidentally overstressed, there is some evidence of fatigue or fire damage or the original specified grade was not supplied. In testing steel girders, it is important to take samples from the (generally thicker) flanges, where the yield stress generally is lower, rather than from the (thinner) webs. It is, of course, desirable to sample at points where the strength requirements are most severe, implying that a fundamental understanding of the structural performance requirements of the facility is required before initiating any in-service inspection program.

To be useful in condition assessment, correlations must be established between the NDE parameter measured (rebound number, pulse velocity, etc.) and the structural property of interest (compression strength, crack size, etc.). These correlations customarily are established by regression analysis and there often is significant statistical scatter in the regression relationships (Snyder, et al, 1992). As an example, the relation between concrete cylinder compression strength and rebound number for a concrete mix with water-cement ratios ranging

from 0.37 to 0.56 and coarse aggregate 0.45 - 0.49 of relative mass is illustrated in Figure 5.1**.

Load testing can be used in some cases to perform a strength evaluation of an existing structure (Hall, 1988). Test loads can be applied using concrete or steel blocks, water, or hydraulic loading devices. During the test, the structure is loaded in stages to a relatively high fraction (say, 75 - 90 percent) of its design strength, the load is held at each stage for a time, and deflections are measured (e.g., ACI 318, 1989). The structure should show no signs of structural damage during the load test, and often a limit is placed on maximum deflection. Following unloading, the recovery of deflection is used to determine whether any permanent plastic deformation has occurred, the occurrence of which might imply nonvisible damage. Load testing should be used only when other methods lead to inconclusive results. It is costly and disrupts the function of the facility; moreover, recent reliability-based studies*** of proof loads indicate that the test load must be above about 90% of the design strength before one can conclude that passage of the load test implies a measurable increase in reliability. At such load levels, there is a high probability that damage (albeit repairable) to the structure will occur. Destructive load testing (to failure) of components is useful only if the components tested are easily repaired, replaced or mass-produced.

5.2 Impact of in-service inspection on time-dependent reliability of navigation structures

Forecasts of time-dependent reliability enable the analyst to determine the time period beyond which the desired reliability of the structure cannot be ensured. Intervals of inspection and maintenance that may be required as a condition for continued operation can be determined from the time-dependent reliability analysis (e.g., Madsen, et al, 1989). Several aspects of ISI/M require statistical treatment.

Nondestructive evaluation methods are imperfect. Intuitively, large defects almost certainly will be detected, while very small defects will almost certainly be missed. The probability of detecting a defect depends on the size of the defect, the NDE method employed, and the training of the operator. The defect detection probability can be expressed in the form of a CDF, as illustrated in Figure 5.2; such a curve can be associated conceptually with each NDE technology and inspector. One common form for probability, $d(x)$, of detecting a defect of size x is given by the exponential distribution,

$$d(x) = 1 - \exp[-c_d (x - x_{min})]; \; x \geq x_{min} \tag{5.1}$$

in which x_{min} = minimum detectable defect (related to the resolution of the NDE device) and c_d = experimentally determined constant. If x is related to the fraction of strength degraded, as in Eqn 3.4, it may be difficult to detect defects that cause less than a 5 to 10 percent decrease in strength, implying that x_{min} = 0.05 - 0.10 for such cases.

** Data provided courtesy of B. Oland, Oak Ridge National Laboratory.

*** Research in progress at Johns Hopkins University by Bhattacharyya and Ellingwood.

Additional information gained during the in-service inspection about the actual strength of the structure depends on the nature of the NDE technology employed. ISI/M changes the CDF or PDF of strength, $f_R(r)$, which is based on prior knowledge of the materials in the structure, construction and standard methods of analysis. This change can be evaluated using Bayesian methods,

$$f_R(r|I) = c\ K(I|r)\ f_R(r) \tag{5.2}$$

in which $K(I|r)$ = likelihood function, $f_R(r|I)$ = updated (posterior) PDF of structural resistance, and c = normalizing constant. The Bayesian updating process is illustrated in Figure 5.2. Scheduled maintenance and repair also may cause the characteristics of the strength to change. The time-dependent reliability analysis then is re-initialized following ISI/M using $f_R(r|I)$ in place of $f_R(r)$.

More generally, suppose that the margin of safety at time t is $M = R(t) - S(t)$ (cf Eqn 4.6). The (instantaneous) failure probability is $P(M < 0)$. Additional information gained about structural performance through ISI/M can be defined by another event, $H < 0$, expressed in terms of structural variables. The revised failure probability following ISI/M is (Madsen, et al, 1989; Jiao and Moan, 1990),

$$P(M<0|H<0) = \frac{P[M<0, H<0]}{P[H<0]} \tag{5.3}$$

For example, if the structure survives a load test with load magnitude $S = q$, then $H = q - R < 0$; Eqn 5.3 becomes,

$$f_R(r|R > q) = \begin{cases} [1 - F_R(q)]^{-1}\ f_R(r); & r \geq q \\ 0 & ; r < q \end{cases} \tag{5.4}$$

A similar approach can be taken if deflection or other response parameter is measured during a structural test. However, the impact of specific ISI/M procedures on time-dependent reliability remains to be examined.

The detection parameters x_{min} and c_d in Eqn 5.1 and the likelihood function $K(I|r)$ in Eqn 5.2 depend conceptually on the NDE technology employed in the in-service condition assessment. However, these parameters presently are unavailable for NDE methods with applications in many civil engineering structures (Malhotra and Carino, 1991; Nasser and Al-Manaseer, 1987; Suprenant, et al, 1992). It seems possible to determine them for flaws in steel weldments, but their determination for a heterogeneous material like concrete is especially difficult. Additional research is required to establish parameters x_{min}, c_d and K for methods likely to be used in ISI/M

for navigation infrastructure. This will require a careful statistical analysis of some limited statistical data that exist from in-service inspections. Nondestructive evaluation techniques, coupled with an appropriate data collection and statistically based assessment framework, are poised to contribute significantly to the development of rational in-service inspection policies.

ISI/M causes the the hazard function, h(t), to change abruptly at the inspection time, t_i, depending on what is learned about the condition of the structure and during inspection and what repair actions are taken. A conceptual illustration of the effect of this process on the hazard function, h(t), is presented in Figure 5.2; the effect of ISI/M is to remove larger defects from the structure and to upgrade its strength, thus reducing its conditional failure rate. As the structure ages, the failure rate increases until another inspection/repair operation occurs. Note that the probability of structural survival during interval (t_1,t_2), given that the structure has survived until t_1, is (cf Eqn 4.10)

$$L[t_2|t_1) = \exp[-\int_{t_1}^{t_2} h(\xi)\, d\xi] \tag{5.5}$$

The integrated effect of h(t) in Figure 5.2 must remain below the target limit state probability, $P_f(t)$.

It should be noted that in-service inspections do not affect the actual probability of failure (unless, of course the inspection is followed by some positive maintenance action). Rather, they enhance the level of confidence in the level of reliability that is estimated and is the basis for the public decision-making.

Structures in navigation facilities generally are too complex for the entire structure to be monitored during its service life. There is a need to: (1) prioritize major components in terms of their impact on the performance of the facility; (2) identify a limited set of potential degradation sites and modes for the current operating spectrum; and (3) develop an in-service inspection plan that is directed toward (but not focussed exclusively on) those limited sites. Some sort of adaptive learning process seems most desirable. An NDE process is envisioned that involves, first of all, inspecting a portion of structure using some noninvasive technique. If damage is found, an additional portion should be inspected, perhaps with a more specialized technique, depending on what is learned at the first inspection. Accessibility and potential hazard to the inspector are two important issues that need consideration. It should be noted also that certain repair operations, such as welding, may actually cause further damage by introducing cracks and tensile residual stresses. Any ISI/M program must represent a compromise between reliability, cost and damage detection and repair effectiveness.

5.3 Life cycle cost analysis

Periodic in-service inspection followed by suitable maintenance may restore a degraded navigation structure to near-original condition. Since inspection and maintenance are costly, there are tradeoffs between the extent and accuracy of inspection, required reliability, and cost. An

optimum inspection/maintenance program might be obtained from the following constrained optimization problem:

$$\text{Minimize } C_T = C_{insp} + C_{rep} + C_f P_f \tag{5.6}$$

$$\text{Subject to } 1 - L(t) \leq P_f \tag{5.7}$$

in which C_T = the total cost, discounted to present worth, C_{insp} = cost of inspection, C_{rep} = cost of repair/maintenance, and C_f = cost of failure, including cost of social disruption due to failure of the facility. Cost C_{insp} depends on the NDE method(s) selected for the inspection.

The process and its impact on the hazard function that is essential to time-dependent reliability analysis are illustrated schematically in Figure 5.3. Several ISI/M strategies can be envisioned; some of these may involve frequent inspection with partial repair while others involve infrequent inspection with thorough repair. An illustration for the concrete slab analyzed in Section 4.3 is presented in Figure 5.4. It is assumed that the limit state probability $P_f(60)$ for a service life of 60 years is to be limited to 4×10^{-4}. In strategy 1, the slab is inspected every 10 years and repaired so as to restore 97% of its initial strength; in strategy 2, the slab is inspected every 20 years and is repaired so as to restore 100% of its strength. Both strategies yield similar reliabilities, and the decision as to which to choose must be made on the basis of discounted present worth value. Clearly, doing nothing leads to an unacceptable limit state probability.

A significant data collection and statistical analysis exercise will be required to determine these costs for navigation facilities or structures. Modern risk analysis based on Eqns 5.6 and 5.7 focuses on both probability (P_f) and consequences (C_f) terms in Eqn 5.4. Low probability events can be very risky if associated with high failure costs (e.g, ASME, 1992). The data to support life-cycle cost criteria in evaluating alternative structural materials and designs are not available at present. Development of these data will require a thorough review of inspection and maintenance information. Nondestructive evaluation provides solutions to the lack of quantitative data. If the NDE information (generally at the local scale) can be incorporated into a rational structural condition and reliability assessment, the long-term economic benefits will be significant.

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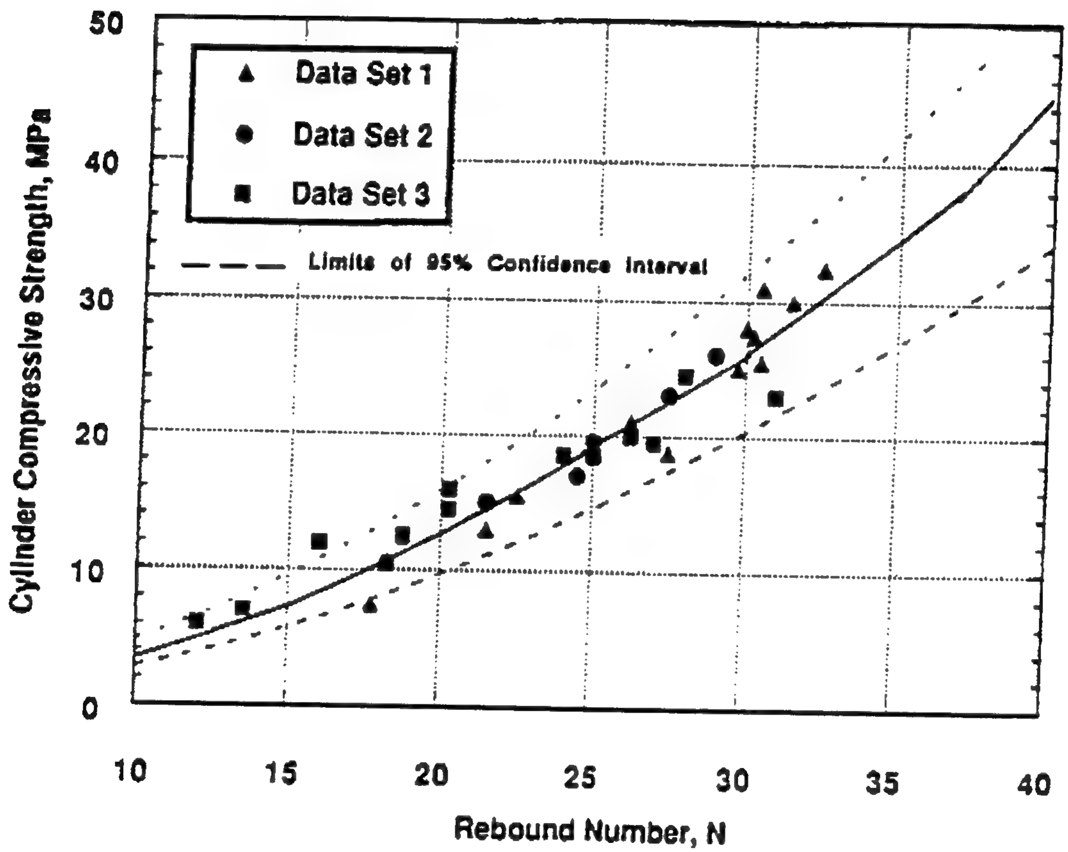

Figure 5.1 - Relation between cylinder compressive strength and rebound number for gravel concrete (Oland, 1993)

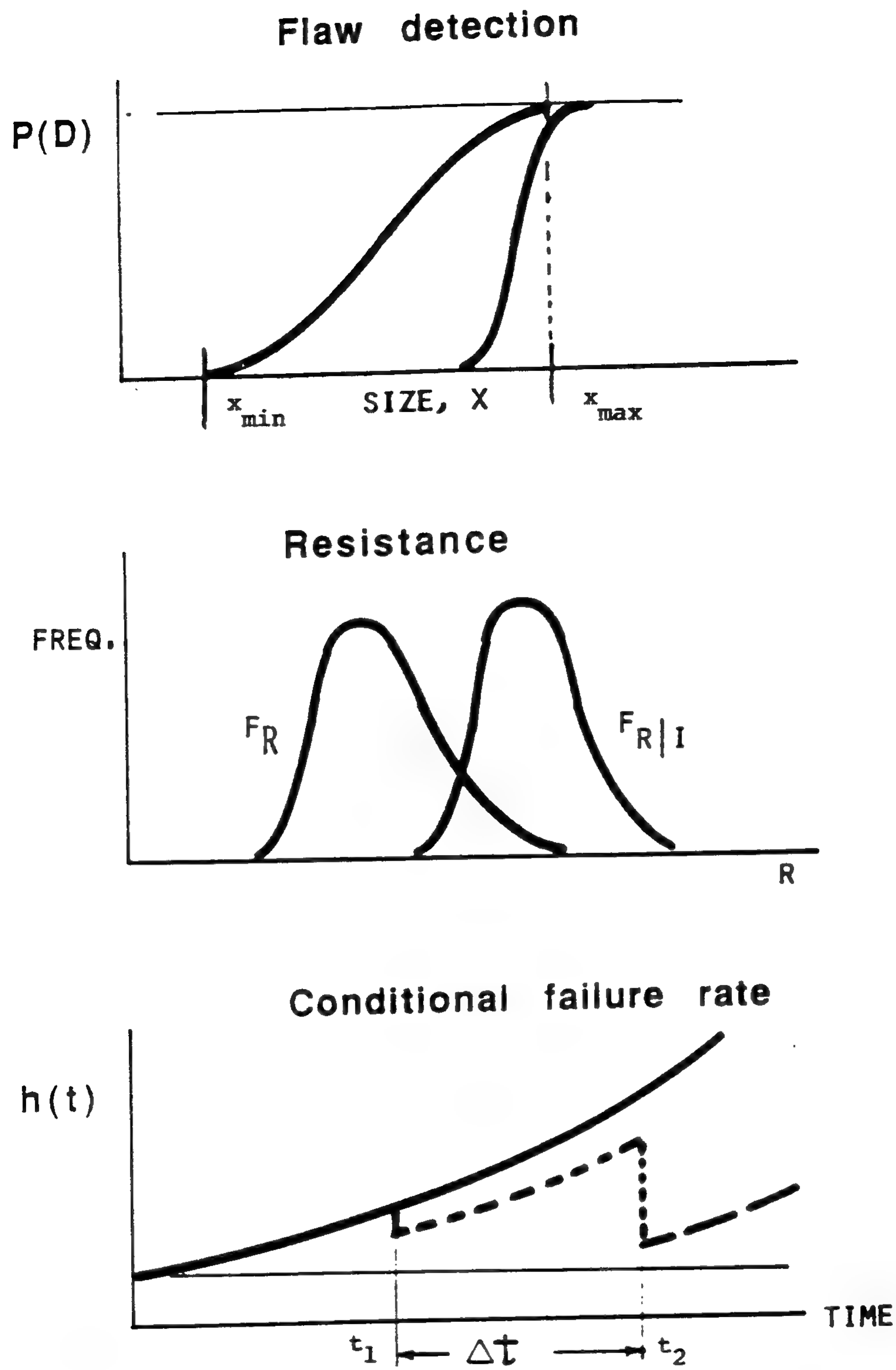

Figure 5.2 - Schematic of in-service inspection and maintenance impact on time-dependent reliability

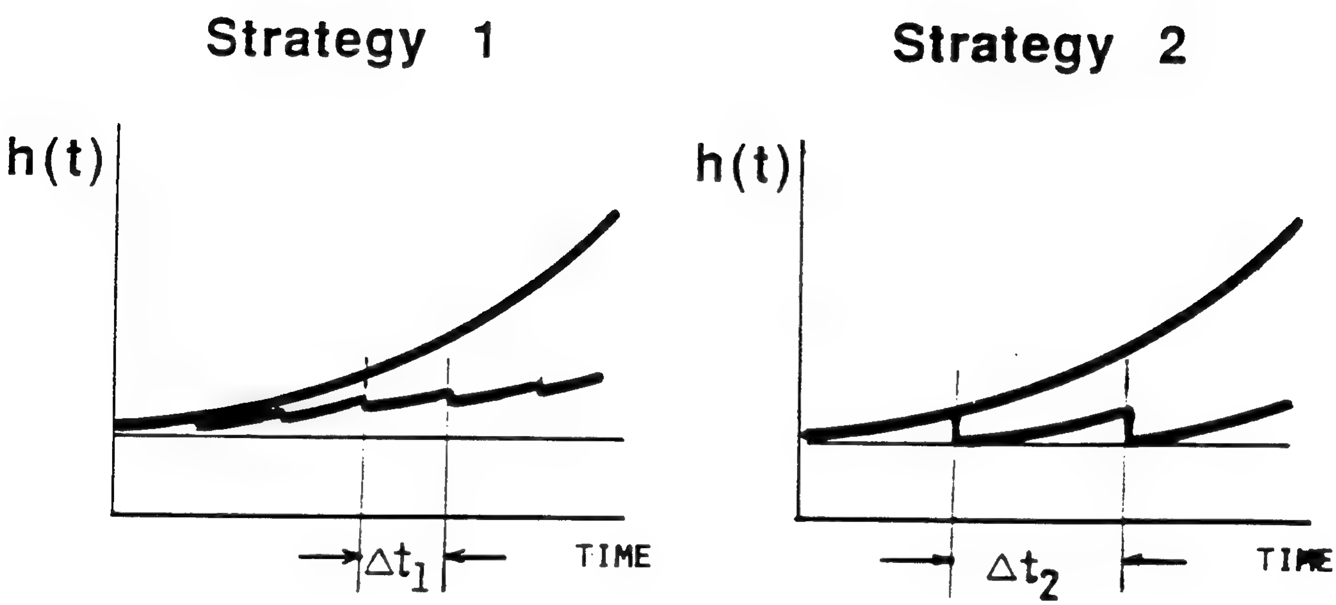

Figure 5.3 - Minimum cost in-service inspection and maintenance

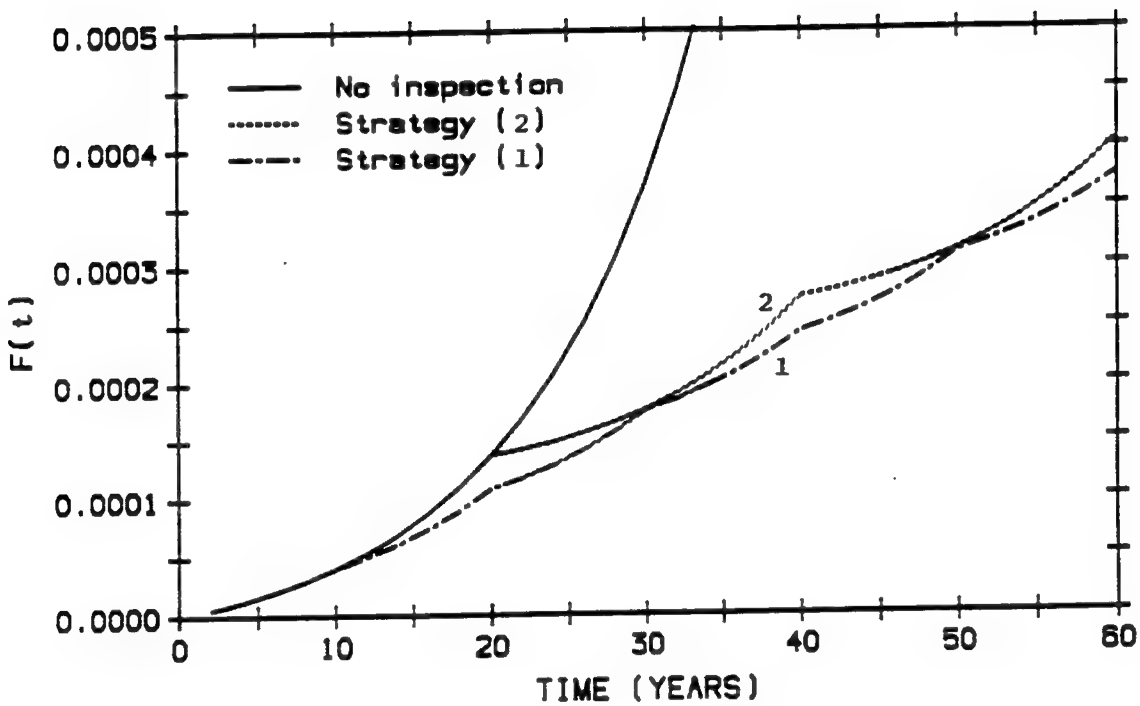

Figure 5.4 - Illustration of limit state probability with and without repair

6. RECOMMENDATIONS FOR FURTHER WORK

This report has reviewed published research on structural deterioration mechanisms caused by operation and aggressive environmental forces and on time-dependent reliability methods that can be used to perform condition assessments of existing structural components and systems. Reliability-based methodologies and data currently are at a state where it should be possible to develop and institute risk-based in-service inspection and maintenance policies for navigation facilities owned and operated by the U.S. Army Corps of Engineers. The methodology leading to these policies should be relatively simple and should be consistent with construction and in-service inspection/maintenance databases currently maintained by the U.S. Army Corps of Engineers. It is believed that the methodology can be completed with a reasonable research effort during the next four years.

Policies should be developed by starting at the system level, with a quantitative statement of expected performance expressed in probabilistic terms. One should work down to the component level. Specific research needs have been collected into six groups below.

(1) Identification of critical structures

Structures found in navigation facilities are complex and may be relatively inaccessible. Experience with other complex engineered facilities suggests that a relatively few structures or components within the facility contribute disproportionately to risk (Hookham, 1991). An effort should be made to identify those dominant contributors, so that the subsequent data collection and reliability modeling efforts can focus on critical structures and components.

(2) Identification of degradation mechanisms and models

It is believed that most relevant degradation mechanisms have been identified in the current report. However, a survey of facility operators should be conducted to determine whether additional mechanisms ought to be considered. Moreover, this survey should obtain some sense of the relative importance of the mechanisms identified, both in terms of structural behavior as well as economic impact on facility performance.

(3) Time-dependent reliability analysis procedures

Methods for reliability analysis of concrete structures subjected to freeze-thaw cycling must be developed and validated. Preliminary findings using simplified models require further validation before being used in decision analysis. Finite element-based analysis of temperature gradients and of moisture diffusion and saturation seem feasible using the transport process models described in Section 3. Finite element analysis can be used to determine structural response due to combinations of operating loads and self-straining thermal effects. The response of a navigation structure to these effects is complex, and unavailable in closed form. Response surface techniques can be used along with finite element analysis to construct a sufficiently accurate limit state model to perform reliability calculations.

Reliability analysis methods to evaluate steel structures subjected to fatigue and/or corrosion must be extended to apply to navigation structures. Techniques for probabilistic

fatigue/fracture analysis must be adapted to consider the unique loading cases and environments found in navigation structures. Efficient numerical analysis tools are required to evaluate joint probabilities, such as those in Eqn 5.3, and conditional failure rates subsequent to inspection. Techniques for analysis of corrosion need to be developed; these will require stochastic characterization of the initiation and active growth phases of corrosion discussed in section 3. Very little research is available on probabilistic aspects of corrosion. Synergistic effects typical of corrosion/fatigue require further study.

(4) Data collection and evaluation

Preliminary work to develop data sources based on accelerated aging tests has indicated that such data may be unreliable when extrapolated to field conditions (Clifton and Knab, 1989). A review of field surveys and in-situ measurements of aging structures is needed is required immediately to identify the necessary descriptive parameters and to provide recommendations for any subsequent data collection in a later phase of the methodology development. Test data to describe fatigue behavior in brackish water typical of many inland navigation facilities is not available at present.

(5) Reliability measures and targets

One of the ingredients of a reliability-based condition assessment and service life prediction is the notion of an acceptable risk or acceptable limit state probability. The selection and interpretation of such a probability, is difficult at the current state-of-the-art, and better estimates are required in support minimum expected cost decision-making.

The target limit state probability should depend on the type of structure, the mode of failure, its relative importance to the facility, the residual life desired, and possible socioeconomic consequences of failure. There appears to be some willingness to accept lower reliabilities for older systems and not to require that older systems have the same reliability as new systems. The desired residual life of the structure depends on its performance requirements and socioeconomic conditions. Risks due to structural failures should be much less than risks from operational failures. Structural failures affect many facility systems simultaneously, and thus the consequences are widespread. Moreover, repair of failed structural components invariably requires that the facility be taken out of service for an extended period to time while repairs are made. Such downtimes may have severe economic consequences.

It is tempting to establish reliability targets by considering comparable and presumably acceptable risks in other human endeavors (Ellingwood, 1994). However, comparisons of failure probabilities of facilities or structures with mortality statistics that have been reported elsewhere are flawed because of differences in the population at risk and the large uncertainties associated with reliability analysis and performance of structures over extended periods of time. Historical data on actual failures provide an incomplete picture of the situation because structural failure events are so rare. Risks associated with engineered structures are very low. There currently is no generally accepted mechanism for comparing risks from diverse hazards with low probabilities, nor is there a comprehensive data base to support such a comparative risk assessment. It has been suggested that civil engineers should concentrate their efforts on managing hazards rather than on assessment of risks (Comerford and Blockley, 1993). Research is required to investigate the

feasibility of developing a framework for comparative risk assesesment. This effort seems particularly important in view of the public nature and visibility of facilities managed by the U.S. Army Corps of Engineers.

The computed risk of a structure or facility is not an attribute of the structure but of the mathematical reliability model used to analyze the structure. It can be used as an attribute to guide decision-making only if the model is dependable (Comerford and Blockley, 1993). Accordingly, an effort must be made to validate risk models of navigation facilities and to determine their limitations for risk analysis and life extension policies. Comparisons then can be made among alternate strategies using probabilistic methods where the uncertainties can be dealt with explicitly and systematically.

The use of fragility modeling as an adjunct to risk management of navigation facilities should be investigated further. A fragility analysis effectively uncouples the probabilistic analysis of system performance from the analysis of the natural or man-made hazard. Focussing on the component or system fragility allows the facility manager insight regarding the dominant contributors to risk without the need to resolve issues associated with the hazard determination, many of which often are difficult or controversial at the current state-of-the-art.

(6) Facility management policies

The reliability-based methodology can be used as a basis for developing rational in-service inspection, re-evaluation, and maintenance programs for structures in navigation facilities. However, the reliability methods are numerically intensive and complex, and likely would be difficult to apply on a case-by-case basis. Accordingly, a set of inspection guidelines should be developed. These guidelines would be reliability-based, but would be couched in a form that would be relatively easy to use in a field office. The guidelines would answer the following questions:

What type of inspections should be done?

What additional analyses should be performed? Simplified or complex? Should they be based on linear elastic analysis or nonlinear analysis?

What should be the requirements of a code for assessment of existing structures?

Should the requalification be done in terms of old or new codes?

What inspection/repair measures are consistent with performance objectives, reliability, and cost?

What sort of documentation should be required?

It is important that the requalification guidelines be made understandable to field engineers. Communication, feedback and control (adaptive learning) are essential ingredients of risk management of facilities in which the characteristics evolve in time (Allen, 1991). Efforts should be made to formalize these processes so as to minimize the real-time learning process.

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13. ABSTRACT *(Maximum 200 words)*

This report has reviewed published research on structural deterioration mechanisms caused by operation and aggressive environmental forces and on time-dependent reliability methods that can be used to perform condition assessments of existing structural components and systems. Reliability-based methodologies and data currently are at a state where it should be possible to develop and institute risk-based in-service inspection and maintenance policies for navigation facilities owned and operated by the U.S. Army Corps of Engineers. The methodology leading to these policies should be relatively simple and should be consistent with construction and in-service inspection/maintenance databases currently maintained by the U.S. Army Corps of Engineers. It is believed that the methodology can be completed with a reasonable research effort during the next four years.

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